Appendix F – Geotechnical Report

Released for Construction
Geotechnical Engineering Recommendations
State Route (SR) 520 Pontoon Casting Facility
Report
Aberdeen, Washington

February 18, 2011

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21-1-21190-012

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ABERDEEN, WASHINGTON

1.0 INTRODUCTION

This report presents the results of subsurface explorations, engineering analyses, and geotechnical engineering recommendations developed for dewatering, foundation excavation, and support of the State Route (SR) 520 Pontoon Casting Facility (PCF) in Aberdeen, Washington.

2.0 PROJECT AND SITE DESCRIPTION

The Washington State Department of Transportation (WSDOT) has contracted Kiewit-General (KG) to construct a casting basin facility to fabricate 33 concrete pontoons within the 55-acre Aberdeen Log Yard property at 400 East Terminal Way, Aberdeen Washington (Figure 1). The property is located within Aberdeen tidelands on the north shore of Grays Harbor near the lower reach of the Chehalis River. The property is bounded by a Port of Grays Harbor facility to the west, the City of Aberdeen Wastewater Treatment Plant to the east, and the Puget Sound & Pacific Railroad mainline and siding to the north.

The purpose of the project is to construct longitudinal, cross, and supplemental stability pontoons that can be put into operation if the existing SR 520 Bridge required emergency replacement. The scheduled project finish date for construction of the PCF and the associated pontoons is May 2014.

The site was previously owned by Weyerhaeuser Corporation, but was purchased by WSDOT in November 2010. KG plans to utilize the entire site to build the casting basin and support facilities.

Historically, two sawmills operated on the site in the last century, but since 1971 the site has been primarily used for log storage. All former sawmill-related structures have been demolished. Between 1971 and 1981, the shoreline was extended to the south through backfill placement with sediments dredged from the Chehalis River, accumulated wood waste, and other fill material.

The site, in general, is relatively flat with several old concrete pads and gravel roads throughout. A distressed pile-supported concrete slab was also observed near the center portion of the site. The ground surface elevation for the main portion of the site where the PCF would be constructed generally varies between about +10 and +15 feet mean lower low water (MLLW) datum. The groundwater elevation is approximately +8 feet.

Based on project drawings, the PCF basin floor is approximately 920 feet long by 190 feet wide, and the bottom of the basin excavation is located at approximately elevation -13 feet. The primary project features are shown in Figure 2 and are discussed below with our general understanding of the geotechnical aspects of the construction sequence.

2.1 Launch Channel and Dolphin Piles

The launch channel slope was evaluated for an inclination of 3 horizontal to 1 vertical (3H:1V) with a rock blanket and for an exposed native slope with an inclination of 5H:1V.

Dolphin piles will be installed in the channel to guide the pontoons as they are launched from the PCF. Dolphin piles would be constructed with 24-inch-diameter by 0.401-inch-thick wall steel pipe piles. The turning dolphins would be constructed with 48-inch-diameter by 1-inch-thick wall steel pipe piles.

2.2 Gate Structure

The construction sequence of the launch channel/gate area will be managed such that the construction of the casting basin gate area will be constructed in the dry within the casting basin excavation, separated from the Chehalis River by a temporary sheet pile cutoff wall at the berm along the riverbank. The existing berm will remain in place until gate construction is complete.

The gate structure consists of a sill, jamb, and bulkhead walls. The gate will be held in place by the jambs and sill. The sill is the bottom submerged horizontal element that connects to the jambs. East and west jamb columns extend upward from the sill. Bulkhead walls extend east and west from the jambs and intersect the slope that continues upward from the casting basin slab elevation. The sill will be supported with 18-inch diameter by 3/8-inch-thick wall steel pipe piles. The jamb and bulkhead walls will be supported by 24-inch-diameter by 0.401-inch-thick wall steel pipe piles. The bulkhead walls would be constructed with concrete panels near the jamb and transition into steel sheet piles near the top of slope. A sheet pile cutoff wall will be constructed below the gate structure to control seepage.

2.3 Casting Basin Excavation

Temporary dewatering wells will be installed so the casting basin excavation can be accomplished. Primary dewatering during construction will be accomplished with temporary dewatering wells and perimeter interceptor drains installed on both sides of the casting basin excavation. If necessary, the wells will be supplemented by temporary sump pumps placed in the excavation area. In general, the groundwater collected from the dewatering system will be treated, as necessary, and re-infiltrated into the ground using an infiltration trench on the east side of the property. This re-infiltration of groundwater would reduce the potential for ground settlement and resulting movements of the Aberdeen Wastewater Treatment Plant facilities. Alternative backup methods for temporary water storage, prior to infiltration, could include diversion to on-site water storage areas. Treatment, on-site storage areas, and infiltration trench locations are shown on the project plans.

2.4 Basin Slab

The 18-inch-thick reinforced concrete basin slab will be supported by 18-inch-diameter driven steel pipe piles spaced on 16-foot centers. The top of slab elevation will be -9 feet. The steel pipe piles will be driven from the existing ground surface prior to excavation for the casting basin. The steel pipe piles will be driven continuously and cut off at the basin elevation with a cutting tool prior to basin excavation.

During operation of the casting basin, groundwater will be collected from underdrains below the casting basin slab and groundwater cutoff trenches in the side slopes. The groundwater collected from the dewatering system will be re-infiltrated into the ground using an infiltration trench on the east side of the property.

2.5 Basin Side Slopes

Cut slopes with an inclination of 2.5H:1V will be excavated to reach the bottom of basin elevation from the existing ground surface. The final top-of-slope elevation will be between +17 and +18 feet. An approximately 4-foot-high toe wall will be constructed at the bottom of the basin side slope. To maintain local stability of the side slopes, considering groundwater seepage, as well as flooding and unwatering of the basin during float-out, a geotextile and 4-foot-thick layer of free-draining, graded, granular filter material consisting of 2 feet of sand and gravel and 2 feet of shot rock will placed on the slope after excavation.

2.6 Gantry Crane Trestle

The gantry cranes that will be used for pontoon construction will be supported by the gantry crane trestle beams and 24-inch-diameter by 0.401-inch-thick wall steel pipe piles.

2.7 Stockpile

Most of the excavated material including organics from the casting basin will be placed on the onsite stockpile in the southwest portion of the site. The inclination of side slopes for the stockpile could range between 3H:1V and 4H:1V depending on soil consistency and placement methods. The extents of the stockpile are shown on the project plans.

2.8 Proposed Parking Lot

An asphalt parking lot will be located on the eastern side of the site. Site access would be obtained over asphalt entry roads from the northeast.

2.9 Precast Laydown Area and General Work Area

Pontoon construction will utilize both precast panels and cast-in-place concrete. The precast concrete panels will be fabricated in the precast laydown areas on the east and west sides of the casting basin. The precast laydown and general work areas would consist of gravel pavement.

The generalized site plan and project features are shown in Figure 2. Locations of project features in Figure 2 are approximate. The project plans show the location of project features.

3.0 PURPOSE AND SCOPE

This report provides the results of our services that were accomplished for the project, including:

- **Mud Rotary Borings**. Observe and sample two mud rotary borings to depths up to 200 feet to evaluate subsurface soil and groundwater conditions, to collect soil samples for laboratory testing, and to perform geophysical testing in the boreholes.
- **Geophysical Testing**. Perform downhole (suspension) geophysical, natural gamma, and resistivity testing at the two boring locations.
- **Groundwater Pumping and Infiltration Testing**. Perform an infiltration test and deep and shallow pumping tests to evaluate the hydrogeologic conditions and dewatering feasibility of the site. Install observation wells and vibrating wire piezometers (VWPs) to obtain groundwater level measurements associated with the pumping and infiltration tests.

- **Geotechnical Laboratory Testing**. Perform laboratory index and strength testing on selected soil samples collected from the field program.
- **Subsurface Characterization.** Utilize subsurface exploration information provided in the Geotechnical Data Report (GDR) included in the Request for Proposals (RFP) documentation and recent explorations to characterize subsurface strength properties and soil type/stratigraphy.
- **Pile Drivability Testing**. Observe a test pile program and dynamic pile testing for several pile size and wall thickness alternatives. The pile driving analysis (PDA) and CAse Pile Wave Analysis Program (CAPWAP) results were used to evaluate the soil resistance of the geologic units, evaluate driving conditions, and develop pile driving acceptance criteria for the proposed piles. The results of these test pile acceptance criteria are summarized in this report.
- **Compressive and Uplift Pile Resistance**. Evaluate the compressive and uplift resistance of the proposed PCF piles.
- **Lateral Pile-Soil Resistance Parameters**. Provide soil parameters to estimate the lateral pile soil resistance at the PCF.
- **Input Soft Rock Reference Time Histories**. Develop spectrum-compatible soft rock time histories for the 1,000-year design ground motion.
- **Site Response.** Perform one- and two-dimensional (1D and 2D) finite-difference site response analyses to model basin soil performance during strong ground shaking.
- **Slope Stability.** Evaluate static slope stability for the basin slopes and stockpile. Seismic and post-seismic response of the basin slopes is evaluated by the 2D finite-difference analysis.
- **Settlement.** Estimate settlement at the site due to fill placement and dewatering and estimate settlement of the proposed stockpile and potential impacts on the project features.
- **Lateral Earth Pressure**. Provide lateral earth and water pressure recommendations for the design of the south basin wall, as well as other walls located inside the basin.
- **Temporary and Permanent Groundwater Dewatering.** The subsurface conditions of the PCF site include shallow perched groundwater, as well as multiple aquifers. Therefore, we provide construction dewatering recommendations to control groundwater inflow from the excavation side slopes, reduce the potential instability of the side slopes, and reduce hydrostatic uplift pressures on the base of the PCF basin slab.
- **Pavement.** Provide asphalt and gravel pavement sections for project traffic/equipment loading.
- **Fill Recommendations.** Provide recommended gradations for the soil materials that will be utilized during construction of the PCF.

- **Geotechnical Instrumentation.** Provide geotechnical instrumentation recommendations for monitoring performance of the PCF during construction and operation.

The main text of the report includes design and construction recommendations and conclusions for the PCF. The appendices of the report contain supplemental information that was utilized to develop the design and construction recommendations and conclusions.

4.0 SUBSURFACE EXPLORATIONS

The subsurface exploration program accomplished for the current study included drilling and sampling two soil borings. Suspension logging for geophysical testing was performed and two VWPs for groundwater monitoring were installed in each boring.

The field explorations were performed between March 29 and April 3, 2010. The borings and VWP installations were accomplished by Gregory Drilling and the geophysical tests were performed by GeoVision Geophysical Services Testing (GeoVision), both under subcontract to KG. The results of the geophysical testing are discussed below and a summary report provided by GeoVision is included in Appendix C. A Shannon & Wilson representative observed the borings. The locations of the explorations were determined by surveying performed by KG. Boring BH-1-10 was moved slightly south of the surveyed location to avoid overhead utility lines. The approximate locations of the explorations completed for this project are shown in Figure 2.

The following sections describe our field exploration program. The exploration logs and a description of drilling and sampling methodology and procedures are provided in Appendix A.

4.1 Borings and Soil Sampling

Two borings, designated as BH-1-10 and BH-2-10, were drilled by Gregory Drilling using a combination of hollow-stem auger and mud rotary drilling techniques to depths of 200 and 195 feet below the ground surface (bgs), respectively. The approximate locations of the borings are shown in Figure 2. The primary objective of these borings was to characterize the soil, retrieve samples of the loose silt and sand for laboratory testing, install VWPs for groundwater monitoring, and perform the geophysical testing.

Soil samples were collected from each boring for geotechnical testing. Soil samples were generally collected at 5-foot intervals to a depth of 185 feet bgs, and below this depth the sampling interval was increased to 10 feet. Relatively undisturbed Shelby tube samples were collected in the loose sandy silt and silty sand material between 25 and 85 feet bgs. Relatively undisturbed Shelby tube samples were extruded and logged. Only index testing was performed on these samples.

Disturbed soil samples were also obtained using the Standard Penetration Test (SPT) method. This test collects a disturbed sample of the soil and provides a measure of the density or consistency of the soil. The number of blows of a 140-pound hammer, free-falling over 30 inches to cause 12 inches of penetration, is termed the Standard Penetration Resistance, or blow count.

4.2 Previous Field Explorations

Previous field and laboratory studies for earlier phases of the project were performed by Landau Associates (Landau) for WSDOT. The locations of the previous explorations are included in Figure 3. The results of these explorations and laboratory testing are presented in a Geotechnical Data Report (GDR) prepared by Landau and dated September 21, 2009. This information was provided in the WSDOT RFP documentation.

4.3 Geotechnical Laboratory Testing

Geotechnical laboratory testing was performed on select soil samples collected from the borings to determine index properties. Geotechnical laboratory testing included the following:

- Visual Classification
- Water Content Determinations
- Grain Size Analyses
- Atterberg Limits (Plasticity) Determinations

Visual classification and water content determinations were generally performed on all samples. The remaining index tests were performed on selected samples. The index tests were performed at the Shannon & Wilson, Inc. laboratory in Seattle, Washington, in general accordance with ASTM International (ASTM) standards. Test procedure descriptions and test results for the index testing are presented in Appendix B. Laboratory test results are also incorporated in the exploration logs included in Appendix A.

5.0 SUBSURFACE SOIL CONDITIONS AND CHARACTERIZATION

We reviewed the results of the explorations located within the general limits of the proposed PCF. We developed additional subsurface profiles using the results of the subsurface explorations presented in the GDR. The positions of these subsurface profiles are shown in Figure 3 and the profiles are included in Appendix D.

In general, the subsurface conditions consist of fill with wood and occasional concrete debris to a depth of about 10 to 15 feet bgs. In some explorations, the thickness of the wood debris appeared to be extensive, while in others it may be limited to less than 1 foot. The depth of wood debris noted in the logs varied between 0 and 23.5 feet, with an average depth of about 11 to 12 feet. The boring logs in the GDR and Appendix A show the elevation and extents of the wood debris. Very soft to medium stiff silt of medium to high plasticity underlies the fill to an elevation of about -60 to -70 feet MLLW.

Very loose to medium dense, silty sand and medium stiff to stiff silt underlie the surficial silt encountered at the site to about elevation -90 to -110 feet MLLW. Based on a review of the subsurface profiles included in Appendix D, the silty sand does not appear to be continuous across the site.

Dense to very dense sand and gravel was encountered below an elevation of about -90 to -110 feet MLLW. Siltstone was encountered at an elevation of about -185 feet MLLW in boring H-08-09.

The project GDR contains the results of numerous in situ and laboratory tests. These tests include SPT, cone penetration tests (CPT), vane shear tests, and pressuremeter tests. Laboratory tests include: 1D consolidation tests, unconsolidated undrained and consolidated undrained triaxial tests, and direct simple shear and cyclic direct simple shear tests. The soil classification and shear strength were compared using the results from the in situ and laboratory tests. These comparisons are included in Appendix D. Soil stratigraphy and strength from these comparisons were used for the analyses described below.

6.0 GROUNDWATER PUMPING TESTS AND ANALYSIS

Shannon & Wilson performed deep and shallow aquifer pumping tests to evaluate the hydrogeologic conditions and dewatering feasibility at the site. We analyzed the pumping test data to estimate the following aquifer characteristics for use in our dewatering evaluation:

- **Hydraulic Conductivity.** The ability of a soil to transmit water. For the purposes of this report, hydraulic conductivity refers to the horizontal hydraulic conductivity.
- **Transmissivity.** The ability of an aquifer to transmit water is equal to the aquifer hydraulic conductivity times the aquifer saturated thickness.

- **Storage Coefficient.** The volume of water released from a unit volume of saturated soil with a unit drop in hydraulic head.

We also performed infiltration testing to evaluate the infiltration capacity of shallow soil at the PCF site. Appendix H provides a discussion of the pumping and infiltration test methods, analysis, and results.

7.0 PILE FOUNDATION DESIGN

7.1 General

The PCF basin slab, gantry crane, and gate structure will be supported on driven steel pipe piles that extend to the dense sand and gravel. The recommendations for pile foundation penetrations and capacities are based on theoretical and empirical data, subsurface conditions encountered at the site, engineering judgment, and experience. In order to confirm our recommendations, a test pile program was performed in April and May 2010. The test pile program consisted of driving a total of eight steel pipe piles at the two locations shown in Figure 2. Each test pile was monitored by Robert Miner Dynamic Testing under subcontract to KG with PDA and CAPWAP performed at the end of driving and at the three- and seven-day re-strikes. A discussion of the test pile program is provided in Appendix E.

The following sections describe the analyses, geotechnical recommendations, and construction considerations for the pile-supported structures at the site.

7.2 Axial Resistance Analyses

Driven pile axial capacities are developed through a combination of side and base resistance. Static axial resistances for the PCF steel pipe piles were estimated based on soil types encountered in the borings, relative densities of the soil as determined by SPT blow count, results of the test pile program, PDA/CAPWAP analyses, and our experience in similar soil and project conditions.

Extreme axial capacities were estimated by considering a loss of side resistance in the potentially liquefiable soils and disregarding the side resistance in the soil overlying the lowest level of potential liquefiable soils. We used static side and base resistances below this layer.

Results of our axial resistance analyses are presented graphically in Figures 4 through 10 in terms of plots of pile penetration versus nominal resistance. Figures 4 and 6 are applicable to piles along the North Profile, located from 350 feet north of the gate structure to the northern extent of the basin. Figures 5, 7, and 8 are applicable to piles along the South Profile, located from 200 feet south of the gate structure to 350 feet north of the gate structure. Figures 9 and 10 are applied to offshore piles, located from 200 feet south of the gate structure to the southern extent of the project site. Since the offshore piles are not designed for the seismic loading case, Figures 9 and 10 only present the Strength Limit Case.

These analyses are applicable to a single pile or pile groups with a center-to-center pile spacing greater than 2.5 diameters. Based on communication with the design team and a review of the project plans, it is our understanding that the design pile spacing for the basin slab, crane trestle, Gate Sill/Jamb/Bulkhead, and Dolphin piles is greater than 2.5 diameters; therefore, axial group reduction factors are not considered.

The recommended penetration elevation to satisfy the required nominal (ultimate, unfactored) resistance can be determined from Figures 4 through 10 using appropriate resistance factors. For Strength Limit compression loading, we recommend a resistance factor of 0.65 for side and base resistance, in general accordance with American Association of State and Highway Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications, 4th Edition, with 2008 Interim Revisions (AASHTO, 2008). This value assumes that dynamic pile testing with signal matching is performed during installation of the production piles. In general accordance with AASHTO (2008), the required number of dynamic pile tests depends on the site variability. Assuming high site variability and the proposed number of piles to support the PCF, we recommend that a minimum of 12 pile dynamic load tests be performed. For Extreme Limit compression loading, we recommend a resistance factor of 1.0 for side and base resistance in general accordance with AASHTO (2008).

A summary of the proposed diameter, pile wall thickness, end condition, and required nominal pile resistance is shown in Table 1. Results presented in Figures 4 through 10 indicate Table 1 required resistances can be achieved by embedment of piles in the dense to very dense sand and gravel layer.

7.2.1 Estimated Settlement

We estimate that the steel pipe piles would experience settlements of about ½ to 1 inch under the proposed factored loads. These settlement estimates do not include the elastic compression of the pile as a result of the applied loading.

7.2.2 Lateral Resistance

Lateral loads acting on the structure from wind, seismic events, and other loadings may be resisted by the lateral resistance provided by the steel pipe piles. The computer programs LPILE^{PLUS} (Ensoft, 2007) and Deep Foundation System Analysis Program (DFSAP) (WSDOT, 2006) may be used to evaluate lateral resistance of driven piles and to calculate the magnitude of deflection, shear, and moment along the pile.

Based on subsurface conditions as interpreted from the subsurface explorations and the results of the 2D finite difference analyses, the recommended parameters for input into the LPILE^{PLUS} and DFSAP programs under static, seismic, and post-seismic/liquefied/softened loading conditions are presented in Table 2.

We recommended that the static parameters be used to evaluate the lateral resistance of the driven piles during static conditions, the seismic parameters be used during earthquake shaking with inertial loads, and the liquefied/softened parameters be used for liquefied/post-seismic loading. The results of the cyclic testing in the low-plasticity silt indicated some excess pore pressure during testing. However, these soils did not achieve liquefaction at over 20 to 25 cycles of shaking. This number of cycles corresponds to the characteristic magnitude for the design ground motion. In our opinion, the elevated pore pressure of the low-plasticity silt would either occur at the end of shaking or not at all, and liquefaction of the sandy site soils would occur after shaking has ended. As shown in Table 2, we recommend that the sand and gravel deposits be modeled with the "Reese Sand" constitutive model, which requires a friction angle and modulus of subgrade reaction. For potentially liquefiable sand and gravel, the friction angle and modulus of subgrade reaction were reduced for the seismic and liquefied loading conditions, as discussed below.

We recommend that the soft silt and soft clay deposits be modeled with the "Soft Clay" constitutive model and the medium stiff to hard silt/clay be modeled with the "Stiff Clay without Free Water." These two constitutive models require an average cohesion and strain at 50 percent max stress (ε_{50}). For silt and clay deposits susceptible to seismic softening, the average cohesion is reduced, as discussed below.

The post-cyclic monotonic direct simple shear test results provided in the GDR did not show a clear, consistent change of the ε_{50} value. In addition, an adjustment of ε_{50} is beyond the state of practice and is not currently documented in the literature. Therefore, the ε_{50} value shown in Table 2 is not adjusted for the seismic or softened loading conditions.

The static soil parameters were estimated based on our review of the consolidated undrained and unconsolidated undrained triaxial tests results, static direct simple shear (DSS) test results, pressuremeter test results, and field vane shear test results, provided in the GDR, CPT correlations (Robertson, 2009; Ladd and Foott, 1974), and our experience with similar soil.

The seismic condition considers the estimated pore pressure ratio developed during ground shaking to reduce the strength of granular soils and seismic loading to reduce the strength of the cohesive soils. The reduced friction angle for the seismic case was estimated using an excess pore pressure ratio of 30 percent and the undrained shear strength was reduced to about 85 percent of the static value, which is generally consistent with excess pore pressures and reduced shear strengths generated in the FLAC model.

The shear strength of the liquefied sands and low-plasticity silts is modeled using a residual friction angle. The softened condition assumes a reduced undrained shear strength of 70 percent of the static shear strength value of the soft clays. In general accordance with the WSDOT Geotechnical Design Manual (GDM), we used the Idriss and Boulanger (2007), Olson and Stark (2002), and Kramer (2008) relationships and an average corrected blow count for each sand/gravel layer to estimate the residual friction angle for the liquefied case. We note that in one instance only the Idriss and Boulanger (2007) relationship provided a residual strength correlation for the blow count being considered. The residual undrained shear strength parameters were estimated based on our review of the cyclic direct simple shear, post-cyclic static DSS laboratory test results provided in the GDR, empirical correlations, excess pore pressure generation in the FLAC models, and our experience with similar soil.

The seismic and liquefied modulus of subgrade reaction was reduced from the static value by the same ratio that the seismic and liquefied friction angles, respectively, were reduced.

Group interaction shall be considered when evaluating horizontal pile movement for piles with center-to-center spacing less than five times the diameter of the pile. When the P-y method of analysis is used, the value of P should be multiplied by a P-multiplier to account for group interaction. Figure 11 shows the AASHTO (2008) recommended p-multiplier for group interaction on laterally loaded piles.

7.2.3 Downdrag

Downdrag loads are created when the soil moves downward relative to the pile, thus transferring load to the foundation. In general, piles most susceptible to downdrag loads are those that pass through a soft, compressible soil and then bear in a stiffer layer. The usual mechanisms that generate downdrag loads are post-construction settlements due to the placement of fill, dewatering, and/or the liquefaction of one or more soil layers. When liquefaction occurs, it results in a sudden settlement of the liquefied layer. As the liquefied layer settles, i.e., as the excess pore pressure dissipates, it creates downdrag loads on the pile, which must be carried by the lower, non-liquefied soil.

We evaluated potential static downdrag loads acting on the piles that support the PCF, gate, and trestle structures as a result of potential settlements that could occur due to the fill placement and dewatering that will be accomplished to construct the facility. Based on the site construction plan, the piles will be installed about 2 to 4 months after fill placement. As discussed in Section 11.2, Grading Settlement Analyses, settlement due to grading and dewatering will occur over a period of about four months. Therefore, some downdrag loads could be imparted to the piles. However, the piles will not be loaded until about eight months after fill placement. After eight months, the consolidation settlement will be substantially complete. When the piles are loaded, they will move down relative to the surrounding soil, creating positive skin friction. Based on these considerations, it is our opinion that static downdrag loads will not act on the piles that support the PCF, the gate structure, and the crane trestle under static loading conditions.

Soil layers that liquefy or soften during seismic loading were evaluated by the finite-difference numerical modeling described below. Liquefaction-induced downdrag loads are estimated using 50 percent of the side resistance of the liquefied layers identified in the finite-difference numerical modeling and 100 percent of the side resistance of the non-liquefied layers lying above the deepest occurrence of potentially liquefiable soil as downward (negative) loads on the pile. The estimated downdrag loads resulting from liquefaction should be added to the factored loads when evaluating the pile resistance required for the extreme event limit state. The downdrag loads are shown as noted in Figures 4 through 8. Figures 4 and 6 are applicable to piles along the North Profile, located from 350 feet north of the gate structure to the northern extent of the basin. Figures 5, 7, and 8 are applicable to piles along the South Profile, located from 200 feet south of the gate structure to 350 feet north of the gate structure. The offshore piles are not designed for the seismic loading case.

7.2.4 Spring Constants

The vertical spring constant, which includes the elastic compression of the pile, for the preferred piles may be determined using the values provided in Table 3. Other spring constants for lateral load and moment resistance may be estimated for the piles using the results of the LPILE^{PLUS} analyses.

7.3 Construction Considerations

The following sections present construction recommendations for driven pipe pile installation.

7.3.1 Installation

The minimum pile driving blow count, stroke height, minimum pile embedment into the dense to very dense sand and gravel bearing layer, and minimum pile driving blow count used to define the top of the dense to very dense sand and gravel are summarized in Table 4 for each of the proposed piles at the project site. All piles should be driven to a minimum tip elevation of -90 feet and to the pile embedment shown in Table 4.

The actual depth of pile penetration achieved will vary depending upon the consistency and relative density of the soil encountered during pile driving. The recommended penetration into the dense sand and gravel and the driving resistance criteria may be modified after the initial production piles are driven and the PDA measurements and CAPWAP analyses are performed.

7.3.2 Pile-driving Conditions

Piles supporting the proposed PCF will be installed through the existing fill and the underlying soft silt and sand deposits into the very dense sand and gravel deposits. Potential obstructions, such as wood and occasional concrete debris and very dense, gravelly material, may be encountered during the installation of the piles through the upper fill material encountered from the ground surface to about 10 to 15 feet bgs. Remedial measures such as pre-drilling and pre-excavation may be required to mitigate the impact of the potential obstructions.

7.3.3 Pile-driving Equipment

All pile-driving equipment should be designed, constructed, and maintained in a manner suitable for the work to be accomplished for this project.

We understand that the piles may initially be driven with a Delmag/APE D-30 diesel pile-driving hammer, hydraulic hammer, or a vibratory hammer to the top of the medium dense sand and gravel encountered at about elevation -60 to -80 feet MLLW. They will then be driven to the required penetration using a Delmag/APE D-46 or D-62 diesel pile-driving hammer.

The basin piles will be driven from the existing ground surface prior to excavating the basin. For installation of the basin piles from the existing ground surface, a specialized internal pipe cut-off tool will be used after the pile is driven full-length from the existing ground surface. Once the pile is accepted, it will be cut off to the required elevation using a specialized cut-off tool that will be lowered inside the pile to the appropriate elevation to cut the pile wall.

7.3.4 Wave Equation Analysis

To establish pile-driving criteria for installation of the preferred piles listed in Table 1, we performed the Wave Equation Analyses for Pile driving (WEAP) using data for the hammer/pile combination to be used in installing the production piles. This method allows evaluation of driving stresses so that an appropriate pile-driving hammer size can be selected to obtain the desired pile resistance with reasonable blow counts and without damaging the piles. This analysis also provides an estimate of the nominal pile resistance for a given pile-driving blow count. All piles should be driven to the driving resistance as determined by WEAP and for required nominal load.

Wave equation analyses were performed on the proposed piles based on subsurface conditions encountered in the explorations and using appropriate pile-driving hammers. The WEAP analyses were performed using the computer program GRLWEAP (Version 2005), which was developed by Goble Rausche Likins and Associates (GRL, 1998). The hammer sizes were selected based on our past experience, input from KG, and performance during the test pile installation.

The WEAP results are presented graphically in Figures 12 through 14 and are summarized in Table 4. Based on the WEAP results for the hammers specified above, we recommend using a steel pipe pile that has a yield stress of least 50 kips per square inch (ksi) for the 18-inch-diameter by 3/8-inch-thick wall, closed-end piles driven for the gate sill and basin slab. A steel yield stress of at least 45 ksi is recommended for the 24-, 30-, and 36-inch-diameter steel pipe piles.

The wave equation analyses results presented are for design purposes only, using assumed hammers and assumed pile data. If the preferred piles and hammer sizes listed above are not selected for construction, we recommend that WEAP be performed utilizing data for the actual hammer/pile combination to be used to install the production piles.

7.3.5 Pile-driving Monitoring

Pile driving should be monitored by taking a continuous driving record of each pile. For this purpose, the pile would be marked in 1-foot increments to facilitate monitoring. As the pile reaches the desired tip elevation, additional 1-inch increments between the 1-foot marks would be required.

The pile-driving record should be complete. The form should have spaces to record hammer stroke (diesel hammers), blows per foot, time, date, reasons for delays, and other pertinent information. In addition, the record should include tip elevation, specified criteria, and initials of inspectors making final acceptance of the pile.

It is often difficult to estimate the energy delivered by diesel hammers with visual observation. The Saximeter, developed by Pile Dynamics, Inc., can be used to record hammer strokes and provide an estimate of the driving energy of diesel hammers. We understand that the Contractor has selected a diesel hammer and, therefore, we recommend that a Saximeter be used during pile driving.

7.3.6 Pile-driving Vibrations

There is the potential for impact of existing nearby structures and utilities from pile-driving-induced vibrations and the resulting settlements. Vibration and settlement considerations are provided in Section 16.

7.3.7 Potential Obstructions

As described previously, and as observed during the test pile program, the proposed piles may encounter wood and occasional concrete debris in the upper materials encountered at the project site. If refusal conditions are encountered, the pile would be extracted, repaired if necessary, the pile location excavated to remove the obstruction, and the pile re-driven. Obstructions may be excavated with a drill or an excavator as the pile driving proceeds through this layer.

7.3.8 Pile Dynamic Testing

The recommendations for pile foundations and, in particular, the recommendations for pile penetrations and resistance are based on theoretical and empirical data, subsurface conditions encountered at the site, and our engineering experience. Additionally, in consideration of the higher resistance factors (RFs) being used to estimate the pile axial compressive resistance, we recommend that at least 12 piles be dynamically tested during driving. The location of these tests will be determined during a pre-construction conference. We recommend that dynamic measurements, using a PDA, be taken, and CAPWAP be performed on each tested pile. The PDA measurements should be taken at the end of initial driving and during re-strike. Restrike of the tested pile should occur a minimum of seven days after the end of initial driving.

7.3.9 Pile Driving Acceptance Criteria

We recommend the pile driving and acceptance criteria presented herein be used by KG field representatives during production pile driving operations. Pile acceptance will include an assessment showing that the driven pile has an estimated minimum nominal axial resistance equal to or greater than the nominal pile foundation demands presented in Table 4. Table 4 specifies the minimum pile driving blow count, minimum stroke height, and minimum pile embedment into the dense to very dense sand and gravel. These criteria may be modified after the initial production piles are driven, and the PDA measurements and CAPWAP analyses are performed.

After the pile is driven to the recommended minimum embedment into the dense to very dense sand and gravel, the continuous pile-driving blow count must be equal to or greater than the values provided in Table 4. If this required driving resistance is not met at the estimated penetration depth, the pile should be driven farther until the continuous pile-driving blow count is achieved.

Piles meeting the continuous pile driving blow count above the minimum penetration into the dense to very dense sand and gravel layer should be driven to a refusal blow count of 15 blows per 1 inch. Pile penetration into the bearing layer less than that shown in Table 4 (including piles that have significant uplift loads) should be evaluated by the geotechnical engineer.

Any interruption in pile driving of more than 30 minutes should be considered a stoppage of continuous driving. The minimum pile-driving blow count criterion should resume after the pile has been driven at least 1 foot after any stoppage of driving.

8.0 ONE- AND TWO-DIMENSIONAL (1D AND 2D) GROUND RESPONSE

8.1 General

We performed two types of site response analyses to estimate the soil response during the design ground motion: 1D equivalent linear total stress analysis and 2D non-linear effective stress analysis. The 1D equivalent linear total stress analysis is a method to estimate site response for soil profiles where pore pressure generation is limited. Although site soil profiles would generate excess pore pressure during strong ground shaking, the 1D equivalent linear total stress analysis would provide relatively higher ground motions as compared to a 1D or 2D non-linear effective stress analysis.

Evaluations of site-specific, non-linear, 2D soil response, including the effects of dynamic pore pressure generation, were performed to evaluate the generation of excess pore pressure, soil softening, and lateral ground displacement effects on the PCF. Two-dimensional models were selected to evaluate the ground response transverse (east-west) and longitudinal (north-south) to the PCF. Appendix G provides a description of the inputs, methods, and results. A summary of the results is presented below.

8.2 Results

8.2.1 One-dimensional (1D) Ground Surface Site Response

Figures G-22 through G-27 show the basin level surface acceleration response spectra for all seven time histories selected for the project in borings BH-1-10, BH-2-10, H-07-09, H-08-09, H-16-09, and H-18P-09, respectively. Also plotted in each figure are the soft rock U.S. Geological Survey (USGS) uniform hazard spectrum and the AASHTO Site Class E spectrum for the project site. Figure 15 plots the geometric mean of the response spectra for each boring. Based on the site response from these borings, the recommended design response spectrum is shown in Figure 15.

8.2.2 Two-dimensional (2D) Effective Stress – Longitudinal

The primary objective for the longitudinal models was to assess the free-field soil movements at the location of the gate structure. Results of horizontal displacements at the gate along the centerline of the basin are shown in Figure 16. The average horizontal displacement is approximately 1.0 feet, moving into the basin. The reason for the movement into the basin is related to the static shear stress developed around and below the sheet pile cutoff in the direction of the basin. The cyclic shear stress pulses from the dynamic loading increase the pore pressures and reduces the shear strength in the sandy zones. As the shear strength in the sandy zones drops and the shear stress demand remains the same or increases, the soil tends to strain toward the basin to relieve the excess shear stress.

Results of horizontal displacements in the longitudinal direction on the outside of the basin are shown in Figure 17. In this case, the average horizontal displacement is approximately 3 feet in the direction of the Chehalis River. Static shear stress adjacent to the gate structure was developed toward the Chehalis River because of the nearby slope bank. As the shear strength in the sandy zones drops and the shear stress demand remains the same or increases, the soil tends to strain toward the Chehalis River to relieve the excess shear stress.

Additional longitudinal simulations were performed to assess the approximate location in which the soil strains transition from moving toward the basin to moving toward the Chehalis River. A summary of the results of these simulations and interpreted values to be used in design are shown in Figure 18. This plot shows that immediately outside of the level ground at the basin slab level, the horizontal displacements begin to shift from moving into the basin to moving toward the Chehalis River. The transition is abrupt and primarily related to the higher ground level on the north side of the gate/bulkhead structure.

8.2.3 Two-dimensional (2D) Effective Stress – Transverse

The primary objective of the transverse simulations was to assess the impacts of dynamic soil movements on the crane wharf structure. Horizontal soil displacements, horizontal pile displacements, pile moments, and pile node angular displacements at the end of shaking are presented in Figures 19 through 28 for sections along the north and south portions of the basin. Note that angular displacements represent the rotation of the pile nodes caused by bending forces and should not be misinterpreted as pile curvature.

Based on the above analysis, the basin slope may experience downslope movements during the design ground motion. Soil movement could occur on the basin and channel sides of the bulkhead given the similar slope configurations and soil conditions. As a result of this slope movement, shear forces could act along the sides of the bulkhead wall, along the top and sides of the bulkhead footings, and along the sides of the embedded sheet pile wall within the upper portion of the zone of slope movement. As a result, for the longitudinal (east-west) bulkhead stability analysis, we recommend that the magnitude of the potential shear force be estimated using an adhesion value equal to 750 pounds per square foot along both sides of the bulkhead wall, along the top and sides of the bulkhead footings, and along the sides of the sheet pile wall.

Based on the estimated depth of the potential ground movements, we recommend that this adhesion be applied to a depth corresponding to about elevation -20 feet. In our opinion, relative movement between the steel sheet pile and steel pipe piles would not occur below this elevation.

9.0 SLOPE STABILITY

We performed stability analyses of the basin slopes, the launch channel, and stockpile using the limit-equilibrium stability program SLOPE/W, Version 7.16, by Geo-Slope International. The Morgenstern-Price method, which satisfies both force and moment equilibrium, was used to calculate factor of safety (FS) values for an optimized failure surface.

The critical circular slip surface was found iteratively by the software and was then systematically and incrementally altered using a numerical optimization process to find the critical slope along the base of each slice of the failing mass. Optimization of a circular failure surface often results in a critical non-circular failure surface. For the basin slope case, we compared the optimized critical failure surface to a non-circular search method failure surface. The resulting failure surface shapes and FSs were similar. As a result, we concluded that searching for a circular failure surface and then optimizing the critical circular failure surface was an appropriate method for this slope and soil geometry.

We evaluated the global stability for the static temporary construction case using undrained strength properties (cohesion) for the cohesive soil and drained strength properties (friction angle) for the granular soil. The global stability of the static long-term case was evaluated using drained shear strength properties (friction angle) for all the soil types. See Section 8 for the analyses related to the basin soil displacement during the design ground motion.

The static drained (friction angle) and undrained (cohesion) soil properties used in these analyses are based on our review of the consolidated undrained and unconsolidated undrained triaxial test results, static DSS test results, pressuremeter test results, and vane shear test results provided in the GDR, CPT correlations (Robertson, 2009; Ladd and Foott, 1974), and our experience with similar soil.

At the launch channel location, we evaluated the global stability for the seismic condition. The seismic undrained soil properties (cohesion) used in the analyses were based on our review of the cyclic DSS provided in the GDR, the excess pore pressure generated in the FLAC model, and our experience with similar soils. The seismic undrained shear strength was reduced to about 85 percent of the static value, which is generally consistent with excess pore pressures and reduced shear strengths generated in the FLAC model. We used a horizontal coefficient of 0.14, which equals about one-half the peak ground acceleration of the design ground motion.

The launch channel soil properties were based on the in situ and laboratory test results performed on soils obtained from the subsurface investigations that extended from 350 feet north of the gate structure to about 200 feet south of the gate structure. The soils located more than 200 feet south of the gate structure appear to exhibit lower strength than the soils nearer the gate. It is our opinion that a potential slope failure of the distant lower strength soils would not adversely impact the gate structure.

For analysis purposes, we used a groundwater elevation consistent with dewatering recommendations at the base of the slab and along the slopes. For locations beyond the extent of the dewatering zone, we estimated the groundwater elevation based on observations made in nearby subsurface explorations and monitoring wells.

9.1 Basin Slope

The slope and toe wall geometry used for this analysis are based on typical basin slope cross sections and site plans. Based on these drawings, we understand the toe wall is approximately 4 feet tall and the basin slope is inclined at a 2.5H:1V slope with a maximum slope height of about 27 feet. We divided the basin into two sections and estimated the global stability for the generalized north and south soil profiles.

For the static construction case, we estimated the static FS for an excavated native soil slope. The static long-term case was evaluated for failure along the geotextile placed on the sand and gravel fill covering the native slope and overlain by shot rock. We used an interface friction angle of 28 degrees which is about 75 percent of the strength of the lower friction angle soil in contact with the geotextile (i.e., the sand underlying the geotextile). This interface friction angle to static soil friction angle ratio is consistent with the upper bound reduction ratio (i.e., more conservative) recommended in Koerner (1998).

We evaluated the global stability of the rapid drawdown case. For this analysis, the clay and silt were modeled using undrained strength parameters (cohesion) and the granular soil (sand and gravel) was modeled using drained strength parameters (friction angle). A water level corresponding to a full basin, about elevation +11 feet MLLW, was applied to the cohesive soil and a water level corresponding to the dewatered condition was applied to the granular soil.

The global stability analyses results are presented graphically in Appendix F and are summarized in Table 5.

9.2 Launch Channel

We evaluated the global stability of a slope height of 29 feet that extends from the generalized ground surface of about elevation 16 feet MLLW to the excavated launch channel elevation of about -13 feet MLLW at inclinations of 3H:1V and 5H:1V. The 3H:1V channel slope alternative was evaluated with about a 6-foot-thick bedding layer and armor rock on top of the slope, and the 5H:1V slope alternative was evaluated assuming the launch channel slope consisted of the existing native soil. The bedding layer and armor rock are described and specified in the Coastal Engineering Report (Coast and Harbor Engineering, 2011).

We also evaluated the stability of the launch channel slope at a distance greater than 200 feet south of the gate. We considered a slope height of 15 feet and an inclination of 5H:1V. The slope extends from the generalized ground surface of about elevation +2 feet MLLW to the excavated launch channel elevation of about -13 feet MLLW. The channel slope was evaluated assuming the launch channel slope consisted of the existing native soil.

The globally stability of the static long-term condition was evaluated using drained strength properties for all soil types. For the static temporary construction condition, the silt and clay were modeled using undrained strength properties and the bedding layer and armor rock were modeled using drained strength properties.

The water level was set at elevation -3.35 feet (Lowest Observed Tide, [LOT]) in the launch channel and then contours the slope to the existing groundwater elevation of +8 feet for the static long-term and construction (rock slope) conditions near the gate. A groundwater elevation of 0 feet was used for the portion of the launch channel greater than 200 feet south of the gate. For the seismic condition, a water elevation of 5.4 feet (mean sea level) was set in the launch channel and contoured along the slope to an existing groundwater elevation of +8 feet. For the construction (native slope) case where the launch channel will be temporarily dewatered, we used a water level of elevation -13 feet in the launch channel and then the water level was contoured to the ground slope to the existing groundwater elevation of +8 feet.

The global stability analyses results are presented graphically in Appendix F and summarized in Table 5.

9.3 Stockpile

The stockpile will consist of soil excavated from the nearby PCF. As the soil is excavated, transported, and deposited in the stockpile, the soil strength will be reduced. The reduced drained strength parameters were estimated using a relationship developed by Kulhawy and Mayne (1990) that relates PI to reduced friction angle and our experience with similar soils. The reduced friction angle was estimated based on the average PI and correlated residual friction angle (Kulhawy and Mayne, 1990). The reduced friction angle and pre-excavation, in situ effective overburden stress were used to calculate a reduced undrained shear strength.

The stockpile geometry used for this analysis is based on project cross sections and site plans. We analyzed the static stability of the stockpile slopes inclined at approximately 3H:1V and 4H:1V. For the purpose of this analysis, we assumed a stockpile height of 20 feet.

The global stability analyses results are presented graphically in Appendix F and summarized in Table 5.

9.4 Results

The estimated global stability FSs for the basin slope, launch channel, and soil stockpile meet or exceed the WSDOT minimum long-term static FS requirements for embankments supporting or potentially impacting non-critical structures (WSDOT GDM Section 9.2.3.1). Both slope conditions analyzed for the launch channel meet or exceed the WSDOT minimum seismic FS requirements for embankment slopes that could impact an adjacent structure. Based on our analyses, the FS for the construction and basin rapid drawdown case is suitable, in our opinion.

The stockpile case with a 3H:1V slope has an estimated long-term FS of 1.3, which equals the minimum FS required by WSDOT, as described above. As a result, the 3H:1V sloped soil stockpile shall be re-analyzed if a soil stockpile greater than 20 feet is proposed for design.

Lateral ground movement of the basin slope during the design ground motion shaking is presented in Section 8.

10.0 LATERAL EARTH PRESSURE

10.1 Lateral Pressures

A sheet pile cutoff wall will be constructed below the gate structure and bulkhead, a bulkhead wall will be constructed as part of the gate structure, and a concrete wall will be constructed at the toe of the basin slopes. The following section presents our recommendations for lateral hydrostatic and earth pressures for retaining and sheet pile cutoff walls.

Lateral earth pressures will act on buried portions of the sheet pile cutoff wall, basin toe wall, and gate bulkhead and sheet pile wall. For walls that are allowed to move at least 0.001 times the wall height, we recommend that active, lateral earth pressures be considered. Active earth pressure diagrams for the gate sheet pile cutoff wall, basin toe wall, and gate bulkhead and sheet pile wall are shown in Figures 29 through 31, respectively.

Hydrostatic pressure will act on the gate cutoff wall when the basin is dewatered and the river is at high tide. The recommended hydrostatic pressure diagram for the gate cutoff wall is shown in Figure 29.

The total earth pressures should be analyzed for seismic loading conditions using a dynamic load added to the static, active earth forces. The dynamic load increase for active pressure conditions for the basin toe wall are shown in Figure 30 and the gate bulkhead and sheet pile wall are shown in Figures 31A and 31C. This load increment should be applied as a trapezoidal load to the wall, with the resultant force acting at 0.6H (where H equals the wall height) from the bottom of the wall. A load increase for seismic conditions is consistent with a pseudo-static analysis using the Mononobe-Okabe equation for lateral earth pressures and a horizontal seismic coefficient of about one-half the soil peak ground acceleration. These pressures assume drained soil conditions behind the wall.

For the gate bulkhead and sheet pile wall, the static active earth pressures shall be used for the static load case (see Figure 31A and 31C), the static active earth pressures and the seismic earth pressure increment shall be used for the seismic loading case (see Figure 31A and 31C), and the post-seismic active earth pressures shall be used for the post-seismic loading case (see Figure 31B and 31D). Figures 31A (static/seismic) and 31B (post-seismic) shall be used for the gate bulkhead and sheet pile wall and Figures 31C (static/seismic) and 31D (post-seismic) shall be used for the outer edges of the basin where there is no bulkhead overlying the sheet pile wall.

General surcharge loading behind walls can be estimated by using the recommendations presented in Figure 32.

10.2 Lateral Resistance

Lateral loads, due to unbalanced lateral earth and water pressures, wind, or seismic forces, could be resisted by passive earth pressures against buried portions of the walls. The passive earth pressure diagram for the gate cutoff (flooded basin and dewatered basin) and gate bulkhead and sheet pile wall are shown in Figures 29 and 31, respectively. These pressures assume the structure extends at least 2 feet bgs. The passive earth pressures shown in the figures are ultimate values and should be reduced by recommended resistance factors for the strength limit case presented in Table 11.5.6-1 of AASHTO (2008-interim).

For the gate bulkhead and sheet pile wall, the static passive earth pressures shall be used for the static load case (see Figure 31A and 31C), and the post-seismic passive earth pressures shall be used for the post-seismic loading case (see Figure 31B and 31D).

11.0 SETTLEMENT ANALYSES

11.1 Stockpile Settlement Analyses

This section presents the results of the settlement analyses performed for the soil stockpile on the west side of the project site. The proposed stockpile will vary in height and could have side slopes that have inclinations up to 3H:1V. For the purpose of this analysis, we assumed a stockpile height of 20 feet. Figure 2 shows the limits of the proposed stockpile. A drainage ditch is located about 50 feet west of the proposed stockpile slope toe (Figure 2).

We used elastic stress distributions and standard one-dimensional consolidation theory to estimate the settlement beneath the stockpile and the drainage ditch. We used the results of the explorations and laboratory testing to develop an idealized subsurface profile for a cross section through the stockpile and drainage ditch. Table 6 summarizes the soil parameters we used in our analysis. We assumed that the stockpile soil has a unit weight of 100 pounds per cubic foot. We also estimated the settlement caused by dewatering the proposed casting basin.

We used two elastic stress distributions to estimate the stress increases due to the stockpile, both from Poulos and Davis (1973):

- Elastic stress under an infinite strip load, for the center of the stockpile; and
- Elastic stress under a linearly increasing infinite load, for the drainage ditch.

Figure 33 shows the estimated ground surface settlement beneath the soil stockpile and the drainage ditch, both pre- and post-dewatering. Figure 33 shows that:

- Settlement beneath the soil stockpile due to soil stockpile placement is about 22 inches;

- Settlement beneath the drainage ditch due to soil stockpile placement is about 2 inches;
- Post-dewatering settlement beneath the soil stockpile due to soil stockpile placement and dewatering is about 27 inches; and
- Post-dewatering settlement beneath the drainage ditch due to soil stockpile placement and dewatering is about 3 inches.

Additionally, based on our analysis, it is our opinion that stockpile settlement impacts on the crane trestle or gate would be negligible.

11.2 Grading Settlement Analyses

This section presents settlement analyses that incorporate the proposed grading (excluding sol stockpile and casting basin) and dewatering operations as discussed below. Figure 34 shows the approximate generalized fill areas and corresponding fill thickness.

For each generalized fill area, we developed a representative soil profile based on nearby soil explorations. The soil profiles extend from the ground surface to about elevation -150 feet (in dense to very dense sand and gravel). Elastic and consolidation soil parameters were estimated based on nearby laboratory and in situ tests. We used 1D consolidation and Atterberg limits tests to estimate consolidation parameters and pressuremeter and shear wave velocity tests to estimate elastic parameters.

We estimated settlement in each generalized fill area using elastic and consolidation theory. We estimated the effective stress increase assuming a uniform load with wide extents and using estimated groundwater drawdown profiles for the steady-state dewatering condition (Appendix H). Based on the estimated effective stress increases and the estimated overconsolidation ratio of the soil, it is our opinion that the state of stress of the subsurface soils in each fill area would be in the recompression zone. Therefore, it is our opinion that the secondary compression of the soil would be relatively small.

Figure 34 shows our estimated settlement ranges for each fill area:

- Area 1 (west of casting basin), 2 to 4 inches;
- Area 2 (northwest of casting basin), 1 to 3 inches;
- Area 3 (northeast of casting basin), 1 to 3 inches;
- Area 4 (far east of casting basin), 1 to 4 inches; and
- Area 5 (near east of casting basin), 3 to 5 inches.

These settlement ranges include elastic compression, consolidation settlements, and the effect of secondary compression. Settlement would begin as the fill is placed and the groundwater table is lowered during dewatering. We estimate settlement would be substantially complete after about eight to twelve months from the start of fill placement and dewatering.

Our analyses do not consider the compressibility and spatial variability of wood debris and organic material that may exist at selected locations across the site. In our opinion, long-term decay/compression of wood and organic material could cause localized settlement at the site. Because much of the site will be paved with gravel, we recommend re-grading the surface and/or placing more fill if settlement occurs.

These settlement estimates should be considered in the design of utilities and other site works that will be accomplished for the project.

12.0 TEMPORARY CONSTRUCTION DEWATERING

Excavation of the PCF will require construction dewatering to control groundwater inflow from the excavation side slopes, reduce instability of the side slopes, and reduce hydrostatic uplift pressures on the base of the PCF basin slab. The subsurface of the PCF site contains shallow perched groundwater, as well as multiple aquifers. The 2010 hydrogeologic field testing performed for this study (discussed in Appendix H) provided data regarding dewatering and infiltration feasibility at the PCF site and included pumping tests in two pumping wells (PW-3-10, bottom elevation -20 feet; and PW-4-10, bottom elevation -50 feet), and infiltration testing in a test pit.

Following the 2010 hydrogeologic field testing program, we worked with the KG team to evaluate alternative construction dewatering approaches:

- Top-of-slope dewatering concept: dewatering wells installed around the perimeter of the PCF at the top of the basin side slopes.
- Toe-of-slope dewatering concept: dewatering wells installed around the perimeter at the bottom (toe) of the PCF basin side slopes.

After preliminary dewatering analyses, the KG team selected the toe-of-slope dewatering concept as the preferred alternative. Thus, we based our dewatering analyses and design recommendations for temporary groundwater control at the PCF site on the toe-of-slope dewatering concept.

12.1 Conceptual Dewatering Model

Our conceptual dewatering model for the PCF site consists of shallow groundwater perched on a silt aquitard, which in turn overlies an upper sand aquifer. Based on available subsurface data and pumping test results, the focus of groundwater control during PCF construction will be the upper aquifer and shallow perched groundwater. Figures 35 and 36 show conceptual dewatering cross sections of the PCF site, including a perimeter cutoff trench partway down the basin side slopes to collect shallow perched groundwater and wells at the toe of the basin side slopes to dewater the upper aquifer (Figure 35) and depressurize the lower aquifer (Figure 36).

12.1.1 Upper and Lower Aquifers

The deep pumping test in PW-4-10 (down to elevation -50 feet) resulted in limited, to no, response in the shallow instrumentation (down to elevation -20 feet) installed at the site, indicating a poor hydraulic connection between the upper aquifer (PW-3-10 pumping test) and the lower aquifer (PW-4-10 pumping test). The shallow pumping tests in PW-3-10 (down to elevation -20 feet), however, resulted in up to 9 feet of drawdown in the shallow monitoring instrumentation. Given proposed excavation base elevations between -13 feet (basin) and -16 feet (gate), the pumping test results indicate that the most efficient approach to construction dewatering at the PCF site would focus on the upper aquifer (approximate elevations -10 to -20 feet), and, providing sufficient saturated aquifer thickness exists beneath the PCF subgrade, dewatering with pumped wells will provide an effective means of lowering groundwater levels in the upper aquifer and a stable subgrade for PCF construction.

Based on the results of the 2010 pumping tests, we do not anticipate that pumping from wells in the upper aquifer (approximate elevations -10 to -20 feet) will dissipate pore water pressure in the lower aquifer (elevation -50 feet). Water-bearing granular layers occurring above approximate elevation -50 feet at the site could create the potential for basal instability of the PCF subgrade. Reducing the hydrostatic head in the lower aquifer(s) present at the site using pumped wells that extend down to elevation -50 feet is recommended to reduce the potential for basal instability of the PCF.

12.1.2 Shallow Perched Groundwater

In addition to the upper and lower aquifers, shallow perched groundwater occurs throughout many areas of the PCF site within mixed fill, logs, and wood debris. The shallow groundwater perches on a shallow layer of clayey silt which appears consistent across the site. Hydraulic properties of the wood/fill layer appear widely variable based on test pits excavated at the site by Landau in 2009 and the test pit excavated in 2010 for infiltration testing by KG (described in Appendix H). Slight groundwater seepage was observed in almost all of the test pits and rapid seepage was observed in several locations. The location of the seepage was generally observed to correspond to the contact between the fill and the underlying native soils. Additionally, the infiltration test pit received 10 gallons per minute (gpm) for many hours at a time during the 2010 infiltration testing, indicating zones of high permeability in the wood/fill layer.

For our dewatering analysis, we have assumed that the three main components of groundwater control during construction at the PCF site include:

- Dewatering the upper aquifer (approximate elevations -10 to -20 feet) and depressurizing the lower aquifer(s) (elevation -50 feet) using large- diameter dewatering wells installed in a perimeter within the PCF at the toe of the basin side slopes.
- Capturing perched groundwater in the wood/fill layer using a perimeter cutoff trench installed partway down the basin side slopes.
- Infiltrating water into a trench east of the PCF to reduce potential drawdown-induced ground settlement at the adjacent water clarifier structures.

12.1.3 Excavation Dimensions and Sequence

Based on project drawings, the PCF basin floor is approximately 920 feet long by 190 feet wide and the bottom of the basin excavation is located at approximately elevation -13 feet. Basin side slopes (2.5H:1V) will extend out of the basin up to a top-of-slope elevation of between 17 and 18 feet. The excavation for the gate structure will extend down to elevation -16 feet on the outboard (river) side of the gate. With an assumed starting groundwater elevation of +8 feet, the required drawdown to achieve groundwater levels at least 2 feet below the base of the excavation for the basin is 23 feet (elevation -15 feet), and for the gate structure is 26 feet (elevation -18 feet).

Based on discussions with the project team, we assume the following excavation sequence and construction activities for the PCF:

- An approximately 200-foot-long temporary sheet pile wall will be installed about 100 feet south of the gate structure. The temporary sheet pile wall will extend to about elevation -40 feet to provide groundwater cutoff during excavation for the gate structure.

- Excavation for the gate structure will be completed; part of gate construction will include the installation of an approximately 360-foot-long permanent sheet pile wall that extends to elevation -42 feet or lower to provide groundwater cutoff for construction and permanent PCF dewatering systems.
- Basin excavation and dewatering operations will progress from the gate northward in four sections, each about 230 feet long. Each 230-foot section will take approximately three to four weeks to complete.

12.2 Groundwater Modeling

We evaluated dewatering well number, spacing, and discharge rates for the PCF construction dewatering system by constructing a transient, three-dimensional numerical groundwater flow model. We used the USGS computer program MODFLOW (McDonald and Harbaugh, 1988), included in the Groundwater vistas (version 5.35) groundwater modeling package (Rumbaugh and Rumbaugh, 2007). We based our model for the PCF construction dewatering system on the stratigraphic sequence estimated by the subsurface profiles as presented in Appendix D, the soil borings and test pits, and the results from the 2010 pumping and infiltration tests.

Table H-4 summarizes the structure and hydraulic soil properties used in the groundwater model, including elevation range, hydraulic conductivity, and assumed soil type of each model layer. The groundwater model domain is about 4,400 feet wide and about 5,900 feet long and consists of 672 rows and 306 columns. The row and column dimensions vary from 100 feet at the outer edges of the model, decreasing to 2 feet in the center of the model where PCF dewatering and infiltration were simulated. Other model details include:

- Constant head boundaries on opposite sides of the model to generate an initial groundwater elevation of about 8 feet in the vicinity of the PCF.
- General head boundaries to simulate the dewatering wells (lowest elevation -20 feet, bottom of the upper aquifer) and the infiltration trench (highest elevation +12.5 feet, approximate ground surface elevation along the east edge of the PCF site).
- A drain boundary to simulate the perimeter cutoff trench and wall boundary to simulate temporary and permanent sheet pile walls at the gate.
- Preconditioned Conjugate-Gradient 2 (PCG2) solver option with a head change convergence criterion of 0.01 foot.

As discussed above, our construction dewatering approach focused on the upper aquifer (approximate elevations -10 to -20 feet). The hydraulic conductivity range of 6 to 12 feet per day ($4x10^{-3}$ to $8x10^{-3}$ feet per minute) used for this unit is based on calibrating the model to pumping time-drawdown data from the 2010 pumping tests in well PW-3-10.

Our analysis includes the following assumptions:

- Initial groundwater elevation: +8 feet
- Upper aquifer: hydraulic conductivity of 6 to 12 feet/day; storage coefficient of 0.01 (dimensionless)
- Lower aquifer: hydraulic conductivity of 7 to 15 feet/day; storage coefficient of 0.01 (dimensionless)
- Shallow perched layer: hydraulic conductivity of 130 feet/day; storage coefficient of 0.2 (dimensionless)
- Required drawdown in upper aquifer: 23 to 26 feet
- Temporary cutoff wall in place: 200 feet long, down to elevation -40 feet, about 100 feet south of and parallel with gate alignment
- Permanent cutoff wall in place: 360 feet long, down to elevation -45 feet, parallel with gate alignment
- Thirty-three dewatering wells along the toe of basin slope fully penetrate the upper aquifer down to elevation -20 feet, with wells spaced 50 feet apart at the gate, expanding to 100 feet apart at the north end of the basin
- Eight out of the thirty-three dewatering wells extend down to elevation -50 feet
- Perimeter cutoff trench located partway down the basin side slope, 4 feet wide, with a base elevation of about 0 feet
- Infiltration trench about 500 feet long, 4 feet wide, and 15 feet deep, aligned about 460 feet east of the basin parallel to the east property boundary

Construction dewatering system components noted above are shown in a dewatering well layout plan (Figure 37).

We note that the results from the 2009 pumping tests (Landau, 2009) indicate hydraulic conductivity values lower than those based on the 2010 pumping tests described in this report. The 2009 pumping tests were performed at elevations below -35 feet in material of lower permeability than that pumped in PW-3-10 (approximate elevations -10 to -20 feet). In our opinion, using the hydraulic conductivity range based on the pumping tests in well PW-3-10 for our dewatering analysis is appropriate, given that the proposed dewatering wells will primarily target the upper aquifer (approximate elevations -10 to -20 feet) during construction.

12.3 Results

We estimate the groundwater discharge from the construction dewatering system could range between 200 and 600 gpm during the early stages of dewatering, decreasing to between 100 and 300 gpm after three months or more of dewatering system operation. If actual subsurface conditions differ (permeability or aquifer thickness) than those assumed, greater or lower dewatering discharge rates may occur.

Tables H-5 and H-6 summarize groundwater modeling results for the low-conductivity and high-conductivity assumptions, including the increasing total number of dewatering wells operating at successive time steps in the transient model. Based on these modeling results, total dewatering discharge rates (dewatering well combined with perimeter cutoff trench discharge) are highest at early stages of dewatering when nearly all the dewatering wells are pumping and the total length of perimeter trench is cutting off perched groundwater.

Construction dewatering groundwater drawdown contour plans are included as Figure H-12 (sheets 1 through 8). This figure shows progressive dewatering and drawdown of the upper aquifer through the basin as dewatering wells become active in groups from the gate area northward at successive time steps in the transient model. The drawdown shown in Figure H-12 is from a layer in the model between elevations -14 and -16 (a portion of the upper aquifer [approximate elevations -10 feet to -20 feet]), which encompasses the anticipated required drawdown elevation of -15 feet throughout the basin.

The modeling results indicate that the eight deep dewatering wells (elevation -50 feet) sufficiently depressurize the lower aquifer(s) to obtain a suitable factor of safety against basal instability of the PCF.

The modeling results also indicate effective drawdown of shallow perched groundwater using the perimeter cutoff trench, and recharge/mounding using the infiltration trench, but are not directly observed in the elevation interval shown in Figure H-12. Dewatering of shallow perched groundwater and mounding from the infiltration trench are captured in shallower layers of the model that represent fill materials overlying native silt. In our opinion, this is an expected result, given our conceptual model (described above) and numerical model layers (summarized in Table H-4), which include a layer of low-permeability silt extending across the site from elevations -10 to +5 feet. Our dewatering recommendations described below include monitoring wells along the east property boundary to evaluate the mitigation provided by the infiltration trench against potential drawdown-induced ground settlement at the adjacent wastewater treatment plant.

12.4 Conclusions and Recommendations

The modeling results indicate that the upper aquifer (approximate elevations -10 to -20 feet) may be dewatered using 33 dewatering wells at the toe of the basin slope, spaced 50 feet to 100 feet apart (Figure 37), provided the assumptions listed above are met. However, a simplified numerical model simulation cannot fully represent the actual variability in soil and groundwater conditions at the site. For instance, interpolation of soil behavior from CPT data (discussed in Appendix D and presented in Figures D-1 and D-2) suggests that the upper aquifer may become thinner and less permeable in the southern third of the basin and around the gate structure. If this is the case, discharge rates may be lower in dewatering wells and additional dewatering components (such as additional or deeper dewatering wells, sumps, trenches, and/or well points) may be needed to achieve groundwater drawdown criteria. In our opinion, given the potential for laterally variable soil and groundwater conditions at the site, 33 dewatering wells on a 50- to 100-foot spacing should be considered a minimum approach for construction dewatering.

The near presence of the basin subgrade to the top of the clayey silt aquitard at approximately elevation -20 feet will likely prevent complete drainage of the upper aquifer (approximate elevations -10 to -20 feet) in some locations. Additionally, it is our opinion that, even with groundwater drawdown criteria achieved in the upper aquifer, fine-grained soils at the PCF site (such as very soft silt with variable clay and fine sand) will potentially retain a high moisture content and will likely remain in a weak and soft condition during excavation. Only time of year (summer and early fall) and lengthy pumping will reduce this problem.

We evaluated the potential for basal instability and hydrostatic uplift pressure in soil units underlying the basin. Our analysis assumes that the dewatering wells have been installed as shown in Figure 37 and function as shown in Appendix H prior to basin excavation. Hydrostatic uplift pressure from the upper aquifer (approximate elevations -10 to -20 feet) would be suitably reduced by the temporary and permanent dewatering systems. In our opinion, the soil column below the basin excavation is sufficiently thick to resist the hydrostatic uplift pressure from the deep gravel layer below an approximate elevation of -100 feet. However, as described above, the lower aquifer(s) occurring above approximate elevation -50 feet at the site could create the potential for basal instability of the PCF. Thus, our dewatering recommendations, listed below, include deep dewatering wells to reduce the hydrostatic pressure in the lower aquifer(s).

We recommend the following for PCF construction dewatering:

- Install 33 dewatering wells at the toe of the basin slope on a 50- to 100-foot spacing as shown in the dewatering well layout plan (Figure 37) in accordance with the dewatering well schematic (Figure H-13); install 25 of the dewatering wells to about elevation -20 feet to extend 3 feet into the silt layer underlying the upper aquifer; install 8 of the dewatering wells to about elevation -50 feet; add dewatering wells as needed based on monitoring well data.
- Install additional VWPs along the center line of the basin (Figure 39) with the tips at elevations -20 and -50 feet as discussed in the Section 16 below and in accordance with the schematic shown in Figure 41; measure groundwater levels in VWPs during construction to evaluate performance of dewatering wells.
- Install temporary monitoring wells along the center line of the basin and at locations around the site perimeter (Figure 39) in accordance with the schematic shown in Figure 41; measure groundwater levels in monitoring wells during construction to evaluate performance of the dewatering wells.
- Install a perimeter cutoff trench partway down the basin side slopes to intercept/collect perched groundwater:
    - Excavate a perimeter cutoff trench (3 to 4 feet wide) that extends to the top of perching silt layer. The elevation of the perching silt varies and is typically about elevation 0 foot;
    - Install a perforated drain in the perimeter cutoff trench and backfill with free-draining sand and gravel; and
    - Route the perforated drain to sump locations installed around the perimeter of the basin.
- Install additional dewatering system components, such as sumps with pumps, as required. Sumps with pumps will likely be required locally to control groundwater that was not intercepted by the dewatering wells or the perimeter cutoff trench.
- Install an infiltration trench (4 feet wide, 15 to 20 feet deep, and about 500 feet long) along the east edge of the site (see Figure 37 for location) to mound groundwater in order to mitigate against potential drawdown-induced ground settlement at the adjacent wastewater treatment plant; backfill the infiltration trench with free-draining gravel.
- Install VWPs along the east property boundary (Figure 39) with the tips at elevations -20 and -50 feet as discussed in the Section 16 below and in accordance with the schematic shown in Figure 41; measure groundwater levels in VWPs during construction to evaluate the performance of the infiltration trench and to verify that sufficient recharge/mounding occurs to reduce the potential for settlement at the adjacent wastewater treatment plant.

- Shannon & Wilson should log and sample the soil encountered during well and infiltration trench installation to observe that the soil conditions represent the formation conditions anticipated and the assumptions used in our dewatering modeling.

Performance criteria for dewatering will be included in project documents, including drawdown limits. Existing and additional monitoring wells in the PCF vicinity should be used to monitor groundwater levels prior to and during construction to evaluate dewatering system performance. We understand that dewatering well installation and operation will begin about a month prior to basin excavation. In the event that the required drawdown criteria are not satisfied, field modifications to the dewatering system will be required and will be determined on a case-specific basis. We recommend allowing a minimum of 30 days of pumping on the recommended wells prior to any system modifications.

We understand that water collected by the temporary dewatering system will ultimately be treated, as required, and discharged to the locations shown in the project drawings.

13.0 PERMANENT DEWATERING SYSTEM

Based on the subsurface conditions encountered near the basin subgrade level, it is our opinion that the basin can be dewatered during the pontoon construction using the recommended 2-foot-thick sand and gravel drainage layer placed below the permanent basin slab, as described in Section 15. Longitudinal perforated drains should be installed at the toe of the basin slopes in the sand and gravel drainage layer below the basin slab and transverse perforated drains should be installed across the basin in the drainage layer. The transverse drains should be connected to the longitudinal drains and should be routed to the sump locations installed around the perimeter of the basin. The design of the drains and spacing has been accomplished by the civil engineer.

The perimeter cutoff trench (described above) should be part of the permanent dewatering system and discharge from the trench should also be routed to the basin sump locations. The sumps should be continuously pumped to maintain lowered groundwater levels so that uplift pressures do not act on the base of the PCF slab.

In addition, we recommend that dewatering wells used during excavation be integrated with the drainage layer to provide additional dewatering capacity in the event that groundwater levels are not lowered in a timely manner during the unwatering cycle of the flooded basin (Figures 35 and 36).

We recommend monitoring groundwater levels in the VWPs installed through the center line of the basin during the unwatering cycles of the facility to evaluate the effectiveness of the dewatering system(s). We also recommend slowing or halting unwatering to allow time for sufficient drawdown in the event that groundwater levels in the upper aquifer (approximate elevations -10 to -20 feet) or lower aquifer(s) (down to elevation -50 feet) are not sufficiently lowered, to mitigate potential base instability of the PCF.

We evaluated potential groundwater discharge rates of the PCF permanent dewatering system by constructing a steady-state MODFLOW groundwater flow model based on the transient model described above for construction dewatering. We applied the same aquifer parameters and model layer assumptions as in the construction dewatering model (summarized in Table H-4), with a drawdown of up to 22 feet in the basin to elevation -13 feet.

We estimate the stabilized, steady-state groundwater discharge from the permanent dewatering system could range from about 100 to 200 gpm. Flow rates should decrease over time as the saturated thickness of the water-bearing soil decreases due to dewatering.

A dewatering groundwater drawdown contour plan for the long-term condition is included as Figure H-14. The drawdown shown in Figure H-14 is from a layer in the model between elevations -12 and -14 (a portion of the upper aquifer [approximate elevations -10 feet to -20 feet]), which encompasses the anticipated drawdown to elevation -13 feet throughout the basin in the long-term condition. Based on this analysis, about 1 foot of drawdown could occur in the upper aquifer up to about 1,900 feet away from the edge of the drainage layer in the steady state condition.

We understand that water collected by the permanent dewatering system will ultimately be treated, as required, and discharged to the locations shown in the project drawings.

We also conducted seepage analyses to evaluate water movement below the PCF gate and sheet pile cutoff wall to estimate exit gradients at the interface between the drainage layer and the underlying native soil. We evaluated potential groundwater seepage conditions at the PCF gate and cutoff wall by constructing a groundwater flow model using the 2D, finite-element seepage analysis program SEEP/W 2007, which is part of the GeoStudio 2007 software package developed by Geo-Slope International (2007).

We developed the steady-state seepage models based on soil and groundwater data collected during previous explorations by Landau (Landau, 2009), and from our 2010 hydrogeologic field testing program. Our evaluation includes the following assumptions:

- Initial groundwater elevation +8 feet
- Chehalis River stage elevation +10 feet
- Drainage layer head elevation -11.5 feet
- Surface elevation -13 feet on outboard (river side) of cutoff wall
- Sheet pile cutoff wall depths varying between elevations -30 and -50 feet
- Saturated soil hydraulic conductivity:
  - Drainage sand: 285 feet per day
  - Slightly silty to silty sand: 6 to 12 feet per day
  - Silt: 0.03 feet per day

Based on the results of these analyses, we recommend that the tip of the sheet pile cutoff wall extend to elevation -42 feet or deeper below the gate structure.. The seepage analyses indicate that extending the sheet pile to elevation -42 feet or deeper results in exit gradients at the interface between the drainage layer and the underlying native soil of 0.1 or less. An exit gradient of 0.1 or less is substantially lower than the maximum exit gradient of 0.5 recommended by the U.S. Army Corps of Engineers (USACE) for levees (USACE, 2000).

14.0 PAVEMENT DESIGN

This section presents design and construction recommendations for the gravel-surfaced and hot-mix asphalt (HMA)-paved areas in the PCF. We understand that the heavy equipment, trucks, and forklifts will be driving over the gravel areas and that lighter-weight vehicles will travel on the HMA-paved areas. The HMA-paved areas comprise the entrance way and a parking lot for passenger cars and light trucks. The HMA paved catch basin access road will be utilized by concrete trucks (Figure 2). An offsite HMA pavement design for two City of Aberdeen street intersections was submitted in a separate memorandum. The pavement subgrade conditions, traffic loading, design methodology, section recommendations, and construction considerations are presented below.

14.1 Subgrade Conditions

The subgrade conditions utilized in the pavement analyses were based on the test pits and borings that were presented in the GDR (Figure 3). Typically, in the upper 15 feet, the subsurface explorations encountered loose to dense silty, sandy gravel and sand fill layer underlain by a layer of wood debris that overlies a soft silt layer. On portions of the site, this fill layer also contains organics and wood debris and has a thickness as great as 7 feet, as observed in the explorations completed at the site. The wood layer was also encountered at the surface in some of the explorations.

14.2 Traffic Load

We understand that various construction equipment including cranes, loaders, forklifts, and trucks will be utilizing the gravel areas during construction of the PCF. The equipment axle loads and other specifications were provided by KG. Based on discussions with KG, we used an axle load of 64 kips from the Hyster H300HD forklift as the design axle load. KG provided 11,000 repetitions for a Hyster forklift axle load to represent the trafficked areas at the site.

For the HMA pavement areas that consist of the entrance way, parking lot, and the casting basin access road, we estimated the traffic load based on the intended use of each area. For the entrance way, we assumed a traffic load of 500,000 equivalent single axle loads (ESALs). This assumed traffic load is the same as the traffic load that was assumed for the off-site pavement (during the duration of PCF construction) (Shannon & Wilson, 2010). The traffic load, as provided by KG, is based on gross vehicle weight of 105,500 pounds for the 8-axle trucks/trailers that are proposed to haul material off the site, 270 daily trips during the year 2011, and 70 daily trips during years 2012 to 2014.

We assumed the parking lots will be used for personally owned vehicles, light trucks, and occasional delivery or service trucks. We assumed an average daily traffic of 350, with 2 percent of the trucks resulting in a traffic load of about 10,000 ESALs.

We understand that concrete trucks will make approximately 1,500 trips on the casting basin access road during the duration of the construction. Therefore, we assumed 5,000 ESALs.

14.3 Design Approach

For the gravel-surfaced areas, due to the axle loads of the construction equipment and the low and variable subgrade strength throughout the PCF, geogrid reinforcement and/or geotextile for separation was considered to reduce the gravel base course thickness. We used the Giroud and Han methodology (Giroud and Han, 2004) to estimate the thickness of the base course. This method considers distribution of stresses, strength of base course material, interlock between the geogrid and base course material, traffic volume, wheel loads, and subgrade strength.

For the HMA-paved areas, we used the AASHTO method (WSDOT, 2005) in accordance with the project requirements and WSDOT Pavement Policy. The AASHTO design method is an empirical design based on actual performance and is a widely used method for HMA pavement design subjected to passenger vehicle and standard truck traffic. It considers the strength of materials and traffic stresses in each layer of the flexible pavement section and the strength of the pavement subgrade.

Based on the above explorations, we found the subgrade conditions and layer thickness to be variable. Therefore, based on discussions with the KG team, we developed pavement sections assuming California bearing ratios (CBRs) of 1 and 10 percent. In our opinion, the above two values of CBR provide a potential range of CBR for the PCF pavement subsurface conditions.

A summary of our inputs for estimating pavement thickness is presented below:

Flexible Pavement Design Parameter	Value
▪ Design ESALs	500,000 entrance way 5,000 casting basin access 10,000 parking lots
▪ Initial Serviceability Index, Pi	4.5
▪ Terminal Serviceability Index, Pt	3.0
▪ Reliability	85 percent
▪ Standard Deviation	0.45
▪ Structural Coefficient – Asphalt	0.44
▪ Structural Coefficient – Aggregate	0.13
▪ Drainage Coefficient, m	1.0
▪ Subgrade M_R	2,500 pounds per square inch (psi) (CBR 1 percent)
▪	10,000 psi (CBR 10 percent)

14.4 Gravel-surface and Hot-Mix Asphalt (HMA) Section Recommendations

Tables 7 and 8 present the recommended section thicknesses for the gravel-surface and HMA pavement sections, respectively. Figure 38 presents the grading areas that correspond to the gravel section thicknesses presented in Table 7. Table 7 presents the section thicknesses for the two CBR values that were assumed for the existing subgrade.

14.5 Pavement Surface Drainage and Subdrainage

Excess water that accumulates in the base course and subgrade layers and does not rapidly drain can reduce the pavement design life and weaken the subgrade support. Water in the pavement can be from surface infiltration through the exposed aggregate surface in the unpaved areas, or pavement cracks in the HMA pavement areas, or from high or perched groundwater.

Therefore, for the HMA paved areas, we recommend constructing drainage ditches or trench subdrains along the pavement edges. The pavement subgrade surface should be graded to drain toward the ditches. The pavement base material should be extended and daylighted into these drainage ditches to ensure drainage continuity. Surface water runoff from the margins of pavement areas should be collected to reduce seepage into the pavement base and subgrade.

For the gravel-surfaced areas, we recommend that the base and ballast surfaces be graded to drain toward the edges of the trafficked areas. Rutting and dislodging of aggregates is expected to occur over time under the wheel paths, especially during the wet or thawing seasons.

15.0 MATERIALS AND CONSTRUCTION CONSIDERATIONS

15.1 Basin Slopes and Cutoff Trench

To maintain local stability of the side slopes considering groundwater seepage, as well as flooding and unwatering of the basin during float-out, a 4-foot-thick layer of free-draining, graded, granular filter material consisting of 2 feet of sand and gravel and 2 feet of shot rock will be placed on the slope after excavation.

A 12-ounce/square yard nonwoven geotextile for drainage filtration should be placed on the sand and gravel filter prior to shot rock layer construction in accordance with WSDOT Standard Specifications Tables 1 and 2 in Section 9-33.2[1] (Geotextile for Underground Drainage Filtration, Moderate Survivability, Class C) with no limitation on the apparent opening size, provided the water permittivity criteria is met. This heavier geotextile will be utilized for drainage and slope protection. In the cutoff trench, a layer of nonwoven separation geotextile should be placed on the exposed native soil prior to placement of the sand and gravel filter soil layer.

We recommend using the following materials for the basin slope and cutoff trench from the WSDOT Standard Specifications:

- Sand and Gravel Filter: Sand Drainage Blanket (Section 9-03.13[1])

- Shot rock: Light, loose riprap (Section 9-13.1[2]) with a maximum size of 14 inches

15.2 Basin Slab Underdrain

A 2-foot-thick free-draining, graded, granular filter material is required beneath the casting basin slab for an underdrain. The basin slab underdrain should consist of a sand and gravel filter soil that corresponds to WSDOT Standard Specifications Section 9-03.13(1) Sand Drainage Blanket. As the basin slab is supported by steel pipe piles, the sand and gravel filter should be graded to a uniform surface and compacted by track-walking with multiple passes of a low-ground pressure bulldozer.

We recommend that a geotextile be placed beneath the basin slab underdrain (2-foot-thick sand and gravel filter). The geotextile would be an 8-ounce/square yard nonwoven geotextile that meets the minimum requirements in accordance with WSDOT Standard Specifications Section 9-33.2(1) Table 3 for soil stabilization.

The soil in the bottom of the basin is subject to loss of strength due to disturbance. If native soil is too soft to support construction equipment without rutting or softening, a working surface that consists of sand and gravel filter soil, ballast, or shot rock could be placed on the native soil to reduce disturbance and provide support for construction equipment. Depending on the subsurface conditions encountered at the bottom of the basin, a layer geogrid(s) may be required for installation of the basin slab underdrain to improve stability and reduce mixing of the sand and gravel filter underdrain soil with native soil.

15.3 General Excavation and Temporary Slopes

To provide safe working conditions and prevent ground loss, excavation slopes should be the responsibility of KG. All current and applicable safety regulations regarding excavation slopes and shoring should be followed. In accordance with the WSDOT GDM, any temporary slopes or shoring should comply with appropriate Washington Administrative Code guidelines.

15.4 General Backfill Placement and Compaction

All backfill should be placed in horizontal lifts and compacted to 90 percent of the maximum dry density (ASTM D 1557). Because of the potential for the subgrade to be relatively soft, it may be difficult to achieve compaction requirements in structural fill lifts near the subgrade. If the subgrade is too soft, loose, or wet to allow adequate compaction, we recommend over excavating below the design subgrade level. If soft, compressible soil is present after overexcavation, a geotextile, geogrid, or shot rock may be required to stabilize the subgrade, before placing backfill.

15.5 Pavement Materials and Construction Considerations

Aggregate top (wearing) course and HMA should be constructed in accordance with WSDOT Standard Specifications for Road, Bridge, and Municipal Construction. HMA should conform to Section 5-04 in the WSDOT Standard Specifications.

The HMA should meet WSDOT Standard Specifications Section 9-03.8 requirements for HMA subjected to less than 3 million ESALs. HMA shall consist of HMA Class ½ inch aggregate (WSDOT Standard Specifications Section 9-03.8), and should be constructed in accordance with the WSDOT Standard Hot-Mix Asphalt Pavement Section.

The top course beneath HMA-surfaced areas would consist of crushed surfacing top course material and should meet the requirements of WSDOT Standard Specifications Section 9-03.9(3) Top Course.

The ballast surface for gravel-surfaced areas would consist of material that should meet the requirements of WSDOT Standard Specifications Section 9-03.9(1) ballast except that the sand equivalent should be a minimum of 30.

The base for gravel-surfaced areas would consist of select borrow material and should meet the requirements of WSDOT Standard Specifications Section 9-03.14(2).

Structural fill that will be used to raise grades beneath the gravel surface and HMA top course should meet the WSDOT Specifications for Common Borrow (Section 9-03.14 (3)). Structural fill should not contain organics or deleterious material.

After stripping is performed, the subgrade of all areas to receive new pavement or gravel-surfacing should be proof-rolled, graded to its design grade, smoothed, sloped, and compacted with a static roller. If loose and/or wet, spongy soil zones are identified in limited areas, the soil should be removed and replaced with ballast and/or compacted select borrow fill depending upon the nature of the subgrade material exposed. This material should then be compacted with a heavy, smooth-drum, static roller.

In areas where separation geotextile and geogrid are to be placed, we recommend that the geotextile be placed on the exposed subgrade that has been cleared, grubbed, and prepared as indicated above. The geogrid should then be placed on top of the separation geotextile. We recommend that the structural geogrid have the following minimum characteristics:

Load Capacity[1]	Units	MD Values	XMD Values
True Initial Modulus in Use	lbs/ft	17,000	27,000
True Tensile Strength at 2 Percent Strain	lbs/ft	250	450
True Tensile Strength at 5 Percent Strain	lbs/ft	550	900

Notes:
[1] True resistance to elongation when initially subjected to a load measured via ASTM International D 6637 without deforming test materials under load before measuring such resistance or employing "secant" or "offset" tangent methods of measurement so as to overstate tensile properties.
lbs/foot = pounds per foot
MD = machine direction
XMD = cross machine direction

Aperture Dimensions:	0.9 to 1.5 inches
Junction Efficiency:	90 percent
Rib Shape:	Square or Rectangular
Rib Thickness:	0.03 inch

Where required by Tables 7 and 8, we recommend that the geotextile be an 8-ounce/square yard nonwoven geotextile that meets the minimum requirements in accordance with WSDOT Standard Specifications Section 9-33.2(1) Table 3 for soil stabilization.

Geotextile and geogrid should be installed according to the manufacturers' installation guidelines.

15.6 Utilities

All utility trenches should be backfilled with select borrow (WSDOT Standard Specifications 9-03.14[2]). Backfill in the pipe zone should consist of gravel backfill for pipe zone bedding (WSDOT Standard Specifications 9-03.12[3]). We anticipate that excavation could be accomplished with conventional excavation equipment, although debris may be encountered. As a result, it may be necessary to increase the thickness of bedding material below utilities to maintain a sufficient thickness above large debris in some areas. Soil exposed at the bottom of the deep trenches may be easily softened or disturbed by construction equipment and operations, especially near the groundwater table. If the subgrade is disturbed due to soft or wet conditions, additional soil excavation below the bedding level is recommended. If the subgrade is relatively dry, the excavated soil can be replaced with additional foundation stabilization material. If wet conditions are present in soft compressible soil after overexcavation, then we recommend placing shot rock, ballast, or select borrow to stabilize the subgrade, before placing foundation stabilization material.

Backfill should be placed in lifts not exceeding 8 inches if compacted with hand-operated equipment, or 12 inches if compacted with heavy equipment. There should be sufficient cover over the pipe, however, so that when heavy compactors are used, the pipe is not damaged during backfill compaction. Backfill above the utility pipe zone should be compacted to 85 and 90 percent maximum dry density (ASTM D 1557), in non-traffic and traffic areas, respectively. Catch basins, utility vaults, and other structures installed flush with the finish grade should be designed and constructed to transfer wheel loads to the base of the structure.

15.7 Wet Weather and Wet Condition Considerations

Most of the soil at the site likely contains sufficient fines to produce an unstable mixture when wet. Such soil is highly susceptible to changes in water content and tends to become unstable and difficult or impossible to proof roll and compact if the moisture content significantly exceeds the optimum. In addition, during wet weather months, the groundwater levels could increase, resulting in seepage into site excavations. Performing earthwork during dry weather would reduce these problems and costs associated with rainwater, trafficability, and handling of wet soil.

Based on our understanding of the construction schedule, rainy periods will generally occur during PCF construction. As a result, we highly recommend that KG review the following considerations to reduce the potential for more difficult earthwork operations during wet weather:

- The ground surface in and surrounding the construction area should be sloped as much as possible and sealed with a smooth-drum roller to promote runoff of precipitation away from work areas and to prevent ponding of water.
- Work areas or slopes should be covered with plastic. The use of sloping, ditching, sumps, dewatering, and other measures should be employed as necessary to permit proper completion of the work.
- If there is to be traffic over the exposed subgrade, the subgrade should be protected from disturbance.
- Earthwork should be accomplished in small sections to minimize exposure to wet conditions. That is, each section should be small enough so that the removal of unsuitable soil and placement and compaction of clean structural fill could be accomplished on the same day. The size of construction equipment may have to be limited to prevent soil disturbance. It may be necessary to excavate soil with a

backhoe, or equivalent, and locate the equipment so that it does not pass over the excavated area. Thus, subgrade disturbance caused by equipment traffic would be minimized.

- Fill material should consist of clean, well-graded, pit-run sand and gravel soil, of which not more than 5 percent fines by dry weight passes the No. 200 mesh sieve, based on wet-sieving the fraction passing the ¾-inch mesh sieve. The fines should be nonplastic.
- No soil should be left uncompacted and exposed to moisture. A smooth-drum vibratory roller, or equivalent, should roll the surface to seal out as much water as possible. Because of the soft subgrades likely present at the site, use of a static roller may be necessary.
- In-place soil or fill soil that becomes wet and unstable and/or too wet to suitably compact should be removed and replaced with clean, granular soil (see gradation requirements above).
- Grading and earthwork should not be accomplished during periods of heavy, continuous rainfall.

The above recommendations apply for all weather conditions, but are most important for wet weather earthwork.

16.0 GEOTECHNICAL INSTRUMENTATION PROGRAM

We recommend a geotechnical instrumentation program be used to document and monitor work performed near settlement and vibration sensitive structures and utilities, dewatering progress, and stockpile deformation. The primary objectives of the geotechnical instrumentation program are:

- Indicate whether or not the construction procedures used are generating surface ground movements and vibration intensities within specified limits.
- Provide early warning of adverse trends and implementation of action levels.
- Provide sufficient data to determine the source of unanticipated ground movement and to plan remedial measures.
- Determine when remedial measures need to be implemented to protect structures, utilities, and other improvements.
- Monitor the degree that protective or remedial measures are limiting deformations and pore pressure responses, and to provide early indication when alternative means of protection may be necessary.
- Provide data for settling legal disputes.

- Confirm design assumptions and provide data that could improve future designs and/or changes to the present design.

We recommend that the geotechnical instrumentation and monitoring program be developed to:

- Survey and document the structural pre-construction of adjacent existing facilities.
- Measure horizontal and vertical movement of existing structures and the stockpile.
- Measure vibration levels resulting from construction activities.
- Monitor opening or closing of existing cracks in adjacent existing facilities.
- Monitor changes to groundwater levels as a result of construction.
- Provide action levels for monitored displacements, pore pressure changes, and other critical measurements.

The following sections provide additional information regarding these proposed activities.

16.1 Pre-construction Survey

Before beginning geotechnical instrumentation installation or construction, a pre-construction survey of accessible buildings, structures, and utilities along the project alignment and within the potential influence distance of proposed construction should be undertaken. The survey should document the existing condition of each facility with diagrams, sketches, photographs, and/or video recordings. The survey records should include, but not be limited to, length and width of existing cracks, number of cracks, indications and locations of past or current seepage, condition of door and window jams, condition of paint, etc. For inaccessible facilities, such as smaller-diameter sewers, a closed-circuit television survey should be preformed. Where applicable, the surveys should be videotaped/photographed and conducted in the presence of representatives of the facility owner, KG, and WSDOT. A formal detailed report for each surveyed facility should be developed and signed by each member of the group.

We recommend that pre-construction surveys be performed for the facilities located along the project's north, east, and west boundaries including the adjacent Aberdeen Wastewater Treatment Plant and Port facilities.

During the pre-construction survey the need for and possible extent of instrumentation and monitoring of site features outside the limits of construction should be determined. Shannon & Wilson should assist KG in review of the pre-construction survey and selection of appropriate instrumentation.

16.2 Geotechnical Instruments

The types, numbers, and locations of the geotechnical instruments depend on the proposed construction methods, sequence, and durations, as well as on the proximity, foundations characteristics, and conditions of adjacent facilities. The instrument types discussed in the following sections should be considered for use in the geotechnical instrumentation and monitoring program. Our proposed geotechnical instrumentation is discussed below and the layout is shown in Figure 39.

16.2.1 Deformation Monitoring Points (DMPs)

Deformation monitoring points (DMPs) are fixed markers (survey hubs, pins, or targets) monitored (in conjunction with standard surveying techniques) to evaluate vertical and horizontal deformations. DMPs are an effective method of monitoring ground and adjacent facility movements to assist with assessing construction-induced impacts. DMPs include near-surface settlement points placed near the ground surface for the purpose of monitoring changes in elevation of existing ground. All settlement points would be monitored by optical or laser survey methods to determine displacements.

Near-surface settlement points (NSPs) consist of settlement rods driven into place to ensure that the rods will move with the soil in which they are embedded. Each settlement rod is protected by a warning stake or bollard to prevent damage from construction traffic. In conjunction with survey equipment, NSPs are used to monitor settlements in unimproved areas, settlement associated with dewatering, and locations adjacent to settlement sensitive structures. Our proposed locations for the NSPs are shown in Figure 39 and a typical section of a NSP is shown in Figure 40. We recommend that the NSPs be located adjacent to proposed project features that will provide a barrier from construction traffic (i.e., vaults, light poles), such that they will not be disturbed as construction proceeds.

All DMPs and NSPs will be monitored by optical or laser survey methods annually with any displacements recorded. At the close of the project a summary of all DMP and NSP displacements will be completed and given to WSDOT for future reference.

16.2.2 Seismographs

Seismographs are instruments that measure vibration intensity and frequency. We recommend that vibration levels from construction activities be monitored at structures located within 100 feet of the area where construction activity is occurring. In general, vibrations should be monitored during the installation of the pipe piles and any sheet piles that are installed with either impact or vibratory hammers, or other construction activities that generate significant vibrations.

Vibrations should be measured in terms of frequency and peak particle velocity (PPV). During construction, seismographs should be placed at the ground surface adjacent to each structure to determine that vibration levels are below the response values. Background vibrations should be recorded for each adjacent structure and at representative ground locations before the start of construction. The response values for allowable PPV should be coordinated with utility and/or structure owners. The magnitude of the response values should consider the nature of the facility, the type of construction, and its existing condition.

16.2.3 Monitoring Wells (MWs)

Monitoring wells (MWs) and VWPs obtain groundwater level measurements associated with the dewatering operations. A typical MW and VWP are shown in Figure 41, and the recommended locations of the MWs and VWPs are shown in Figure 39. The primary purpose of the MWs and VWPs is to observe groundwater drawdown around the site for correlation to settlement observed by the NSPs. The groundwater measurements will also provide an early indication of future potential ground settlements. That is, the pore pressure changes will generally occur before ground settlement would be observed, considering the fine-grained nature of the foundation soils. Additionally, the VWPs beneath the basin slab would be permanent and used to observe pore pressure during the unwatering cycles. Dataloggers can be connected to the VWPs, and water level loggers can be installed in the MWs to obtain groundwater level readings at closely spaced time intervals without the need for manual surveying. KG may elect to install dataloggers and water level loggers in select MWs or VWPs near settlement sensitive facilities.

16.3 Monitoring Frequency

Monitoring frequency would vary widely for each of the instrument systems and for each category of construction. DMPs, MWs/VWPs, and seismographs should be installed and a minimum of four readings, ideally at least one week apart, should be obtained before the start of construction to provide a baseline.

A typical monitoring frequency for DMPs is once-daily visual monitoring of points within 100 feet of pile driving operations. The visual monitoring, performed by KG, should include observations such as ground and/or structure cracking, gradual ground depressions and slopes, pavement cracking or settlement, and similar indications of ground, structure, and pavement distress.

When construction has been completed and the permanent dewatering system is functioning, the monitoring frequency can be increased to once a month depending upon the results of the MWs and VWPs readings. If groundwater levels and pressures continue to change over the one-month period, the frequency of the survey measurements should be increased to weekly. All DMPs monitored during pile driving should be monitored at least weekly until those operations are complete.

We recommend continuous seismograph monitoring for vibration-causing activities within 10 feet of cast-iron water mains, within 20 feet of other pipelines, and within 100 feet of other structures.

All MWs and VWPs should be monitored weekly until the construction of the basin is completed. Some of the VWPs are temporary for use during construction. The VWPs beneath the basin slab would be permanent and used to observe pore pressure during the unwatering cycles. The MW monitoring frequency could be decreased to bi-weekly when the permanent dewatering system is in operation. This frequency should be increased to daily during the flooding and unwatering cycle of the basin. The VWPs beneath the basin slab would be permanent and used to observe pore pressure during the unwatering cycles. The VWPs beneath the basin slab would be monitored on an hourly during unwatering cycles.

16.4 Response Values

Response values should be established for structures, utilities, and other critical features prior to the start of construction. These response values would be based on the condition of the structures and utilities and the baseline monitoring data. The response values typically include "threshold" and "limiting" values. The threshold values represent a level of movement that warrants attention. If the instruments indicate that the threshold values have been experienced, remedial measures should be prepared in order to mitigate the vibration, movement, or adverse pore pressure changes that are occurring. Threshold values are typically some percentage of limiting values. If the instruments indicate that the limiting value has been experienced, remedial measures should be implemented immediately or construction suspended to prevent adverse impacts to the structures being monitored.

16.5 Data Reduction and Reporting

Baseline measurements should be obtained as early as possible prior to the beginning of construction. Baseline data is useful for establishing response values and assessing the need for implementing mitigation measures, as well as for resolving potential disputes, especially with respect to the impacts of construction on adjacent structures.

Since the collected and reduced data may be critical to assessing performance, the data must be reported within a few hours. Therefore, we recommend that data be shared verbally within eight hours of the readings being collected.

Due to the quantities of data that could be collected on a daily basis, only the values that approach the threshold need to be reported by KG to the engineer. The communication should include a summary of the construction activities performed during the monitoring period in the vicinity of the instrumentation. This communication will allow the PCF to perform as designed.

Shannon & Wilson has prepared Appendix I, "Important Information About Your Geotechnical/ Environmental Report," to assist you and others in understanding the use and limitations of our reports.

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Items related to hydrology and dewatering were prepared by or prepared under the direct supervision of Larry M. West, L.H.G.

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TABLE 1
SUMMARY OF FOUNDATION PILES AND REQUIRED RESISTANCES

Location	Pile Diameter (inch)	Pile Wall Thickness (inch)	End Condition	Nominal Compression Resistance (kips)	Nominal Tensile Resistance (kips)
Basin Slab	18	3/8	Closed	860	-
Crane Trestle	24	0.401	Closed	1,100	300
Bulkhead Trestle	24	0.401	Closed	1,000	-
Gate - Sill	18	3/8	Closed	630	-
Gate - Jamb	24	0.401	Open	530	550
Gate - Bulkhead	24	0.401	Open	630	480
Dolphin (Plumb)	24	0.401	Open	Controlled by Lateral Resistance	
Turning Dolphin (Plumb)	48	1	Open	Controlled by Lateral Resistance	

TABLE 2
RECOMMENDED GEOTECHNICAL PARAMETERS FOR DEVELOPMENT OF L-PILE P-y CURVES

Location	Top Elevation (feet)	Bottom Elevation (feet)	Ground Slope (% grade)	Soil Model	Total Unit Weight, γ (pcf)	Effective Unit Weight, γ' (pcf)	Average Cohesion, c (psf)			Friction Angle, φ (degrees)			Modulus of Subgrade Reaction, k (pci)			Static/Seismic Softened Strain at 50% Max Stress, ε_{50} for Clay Model	Soil Modulus for Clays [1] Es, (ksi)
							Static	Seismic	Softened	Static	Seismic	Liquefied	Static	Seismic	Liquefied		
350 feet north of gate structure to northern extent of basin	H + 5	H = varies [2]	22	Reese Sand	130	130 [4]	-	-	-	34	34	34	70	70	70	-	-
	H = varies [2]	-11	22	Soft Clay	95	33	500	425	350	-	-	-	-	-	-	0.02	0.23
	-11	-20	-	Reese Sand	120	58	-	-	-	32	24	15	75	55	35	-	-
	-20	-35	-	Soft Clay	95	33	800	675	550	-	-	-	-	-	-	0.02	0.37
	-35	-40		Stiff Clay w/o Free Water	100	38	1100	1100	1100	-	-	-	-	-	-	0.015	0.51
	-40	-60	-	Reese Sand	120	58	-	-	-	30	19	7	50	33	15	-	-
	-60	-75	-	Stiff Clay w/o Free Water	100	38	1100	1100	1100	-	-	-	-	-	-	0.015	0.51
	-75	-105	-	Stiff Clay w/o Free Water	105	43	1500	1500	1500	-	-	-	-	-	-	0.01	0.70
	-105	-110	-	Reese Sand	130	68	-	-	-	36	36	36	90	90	90	-	-
	-110	-160	-	Reese Sand	135	73	-	-	-	38	38	38	125	125	125	-	-
350 feet north of gate structure to 200 feet south of gate structure	H + 5	H = varies [2]	22	Reese Sand	130	130 [3]	-	-	-	34	34	34	70	70	70	-	-
	H = varies [2]	-11	22	Soft Clay	95	33	500	425	350	-	-	-	-	-	-	0.02	0.23
	-11	-15	-	Soft Clay	95	33	550	475	400	-	-	-	-	-	-	0.02	0.26
	-15	-20	-	Reese Sand	120	58	-	-	-	32	24	15	75	55	35	-	-
	-20	-35	-	Soft Clay	95	33	800	675	550	-	-	-	-	-	-	0.02	0.37
	-35	-55	-	Stiff Clay w/o Free Water	100	38	1100	1100	1100	-	-	-	-	-	-	0.015	0.51
	-55	-70	-	Reese Sand	120	58	-	-	-	30	19	7	50	33	15	-	-
	-70	-75	-	Stiff Clay w/o Free Water	100	38	1100	1100	1100	-	-	-	-	-	-	0.015	0.51
	-75	-95	-	Stiff Clay w/o Free Water	105	43	1500	1500	1500	-	-	-	-	-	-	0.01	0.70
	-95	-105	-	Reese Sand	130	68	-	-	-	34	34	34	80	80	80	-	-
	-105	-160	-	Reese Sand	135	73	-	-	-	38	38	38	125	125	125	-	-

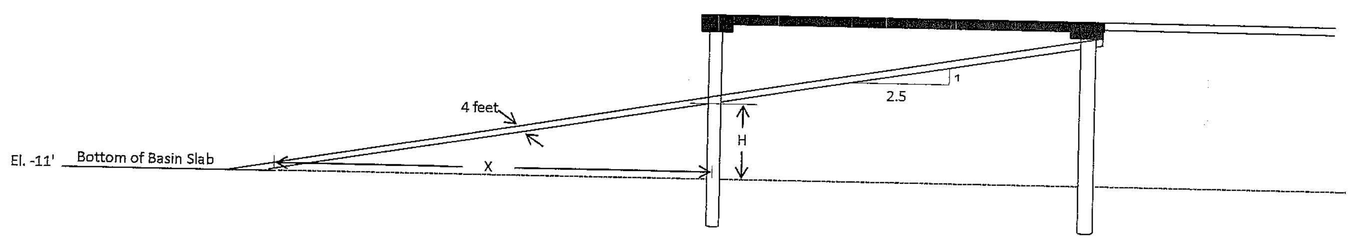


TABLE 2

RECOMMENDED GEOTECHNICAL PARAMETERS FOR DEVELOPMENT OF L-PILE P-y CURVES

Location	Top Elevation (feet)	Bottom Elevation (feet)	Ground Slope (% grade)	Soil Model	Total Unit Weight, γ (pcf)	Effective Unit Weight, γ' (pcf)	Average Cohesion, c (psf)			Friction Angle, φ (degrees)			Modulus of Subgrade Reaction, k (pci)			Static/Seismic Softened Strain at 50% Max Stress, ε_{50} for Clay Model	Soil Modulus for Clays [1] Es, (ksi)
							Static	Seismic	Softened	Static	Seismic	Liquefied	Static	Seismic	Liquefied		
200 feet south of gate structure to southern extent of project site	-11	-20	-	Soft Clay	95	33	200	NA	NA	-	-	-	-	-	-	0.02	0.09
	-20	-35	-	Soft Clay	95	33	300	NA	NA	-	-	-	-	-	-	0.02	0.14
	-35	-65	-	Stiff Clay w/o Free Water	100	38	800	NA	NA	-	-	-	-	-	-	0.015	0.37
	-65	-75	-	Stiff Clay w/o Free Water	100	38	1000	NA	NA	-	-	-	-	-	-	0.015	0.47
	-75	-95	-	Stiff Clay w/o Free Water	105	43	1500	-	-	-	-	-	-	-	-	0.01	0.70
	-95	-105	-	Reese Sand	130	68	-	-	-	34	-	-	80	-	-	-	-
	-105	-160	-	Reese Sand	135	73	-	-	-	38	-	-	125	-	-	-	-

Notes:

1. Es = 0.465 * Su(ksf), provided for "Buckling and Lateral Stability" calculation as required by Section 10.7.3.13.4 in American Association of State Highway and Transportation Officials (AASHTO, 2009).
2. Vertical height of soil should be calculated using the following equation, H = (1/2.5) * X, where H is the height above base of slab elevation (-11 feet) and X is the horizontal distance from the toe of the slope (see Page 1 of 2).
3. Use an effective unit weight of 68 pcf when basin is submerged.

pcf = pounds per cubic foot
pci = pounds per cubic inch
psf = pounds per square foot
ksf = kips per square foot
ksi = kips per square inch

TABLE 3
RECOMMENDED VERTICAL SPRING CONSTANTS FOR STEEL PIPE PILES

Pile Type	Location	Pile Diameter (inch)	Wall Thickness (inch)	End Condition	Recommended Vertical Spring Constant, K, (kip/inch)
Steel Pipe	Basin Slab	18	3/8	Closed	500 - 600
	Gate Sill				
	Crane Trestle	24	0.401	Closed	700 - 800
	Bulkhead Trestle	24	0.401	Closed	700 - 800
	Gate Jamb and Bulkhead	24	0.401	Open	600 - 700
	Dolphin (Plumb)	24	0.401	Open	not analyzed
	Turning Dolphin (Plumb)	48	1	Open	not analyzed

TABLE 4
PRELIMINARY PILE DRIVING CRITERIA
(May be Revised Based on Production Pile PDA Measurements)

Location	Pile Diameter (inch)	Pile Wall Thickness (inch)	End Condition	Hammer Type	Nominal Compression Resistance (kips)	Nominal Tension Resistance (kips)	Gravel Contact Blow Count[1] (blows/foot)	Continuous Pile Driving Blow Count[2] (blows/foot)	Minimum Penetration Into Gravel[2,3] (feet)	Minimum Stroke (feet)	Maximum Compression Stress (ksi)
Basin Slab	18	3/8	Closed	D-46	860	-	30	100	20	8.0	40
Crane Trestle	24	0.401	Closed	D-62	1,100	300	35	100	10	8.8	37
Bulkhead Trestle	24	0.401	Closed	D-62	1,000	-	35	80	5	8.6	37
Gate - Sill	18	3/8	Closed	D-46	630	-	30	60	5	7.7	38
Gate - Jamb	24	0.401	Open	D-46	530	550	12	24	30	6.7	27
Gate - Bulkhead	24	0.401	Open	D-46	630	480	12	30	30	6.9	28
Dolphin (Plumb)	24	0.401	Open	Vibratory Hammer	-	-	-	-	Controlled by Lateral	-	-
Turning Dolphin (Plumb)	48	1	Open	Vibratory Hammer	-	-	-	-	5	-	-

Notes:

[1] The minimum blow count used to define the top of the dense to very dense sand and gravel layer.

[2] Acceptance criteria is based on a continuous pile driving blow count and a minimum penetration into gravel.

[3] Recommendations for minimum embedment into gravel are for axial resistance only. Additional embedment may be required for lateral resistance.

ksi = kips per square inch

TABLE 5
SUMMARY OF GLOBAL STABILITY ANALYSES RESULTS

Location	Geometry	Factor of Safety				Figure No.
		Static				
		Construction	Long-term	Rapid Drawdown	Seismic	
Basin Slope North [2]	Slope = 2.5H:1V Height = 27 feet	1.2	1.3	1.4	See Note 1	F-1
Basin Slope South [3]	Slope = 2.5H:1V Height = 27 feet	1.2	1.3	1.4	See Note 1	F-2
Basin Toe Wall	Slope = 2.5H:1V Height = 27 feet, Geotextile on slope	N/A	1.3	N/A	See Note 1	F-1
Soil Stockpile	Slope = 3H:1V Height = 20 feet	1.4	1.3	N/A	N/A	F-3
	Slope = 4H:1V Height = 20 feet	1.6	1.6	N/A	N/A	F-4
Launch Channel	Slope = 3H:1V (native soil) Height = 29 feet	1.4	N/A	N/A	N/A	F-5
	Slope = 3H:1V (6 foot gravel cover) Height = 29 feet	1.5	1.3	N/A	1.1 to 1.2	F-5
	Slope = 5H:1V (native soil) Height = 29 feet	2.2	1.3	N/A	1.2 to 2.1	F-6
	Slope = 5H:1V (native soil) Height = 15 feet, Offshore Profile[4]	1.9	1.4	N/A	N/A	F-7
WSDOT Minimum Required for non-critical structures		N/A	1.3	N/A	1.1	-

Notes:

1. See Section 8.2.3 in the main text for discussion of lateral ground movement of the basin slope during the design ground motion shaking (i.e., seismic and post-seismic stability).
2. North profile extends from 350 feet north of the gate to the north extent of the basin.
3. South profile extends from 200 feet south of the gate to 350 feet north of the gate.
4. Offshore profile extends from 200 feet south of the gate to the south extent of the site.

H:V = horizontal to vertical, N/A = Not applicable, WSDOT = Washington State Department of Transportation

TABLE 6
SOIL CONSOLIDATION PARAMETERS FOR SOIL STOCKPILE SETTLEMENT ANALYSES

Layer Top Depth (feet)	Layer Bottom Depth (feet)	Effective Unit Weight (pcf)	Overconsolidation Ratio	Compression Coefficient, $C_{c,\varepsilon}$	Recompression Coefficient $C_{r,\varepsilon}$
0	15	80	2.5	0.22	0.024
15	30	40	2.5	0.21	0.024
30	45	40	1.5	0.21	0.024
45	60	40	1.5	0.21	0.024
60	75	60	1.5	0.16	0.014
75	95	60	1.5	0.18	0.014
95	115	60	1.5	0.19	0.017

Note:

pcf = pounds per cubic foot

TABLE 7
GRAVEL PAVEMENT SECTIONS

Material	Layer Thickness (inches)	
	Subgrade CBR: 10 Percent	Subgrade CBR: 1 Percent
	Number of Passes[1]: 11,000	Number of Passes[1]: 11,000
Ballast Surface	6	6
Select Borrow Base	12[2]	30
Geogrid and Geotextile on the Subgrade	No	Yes

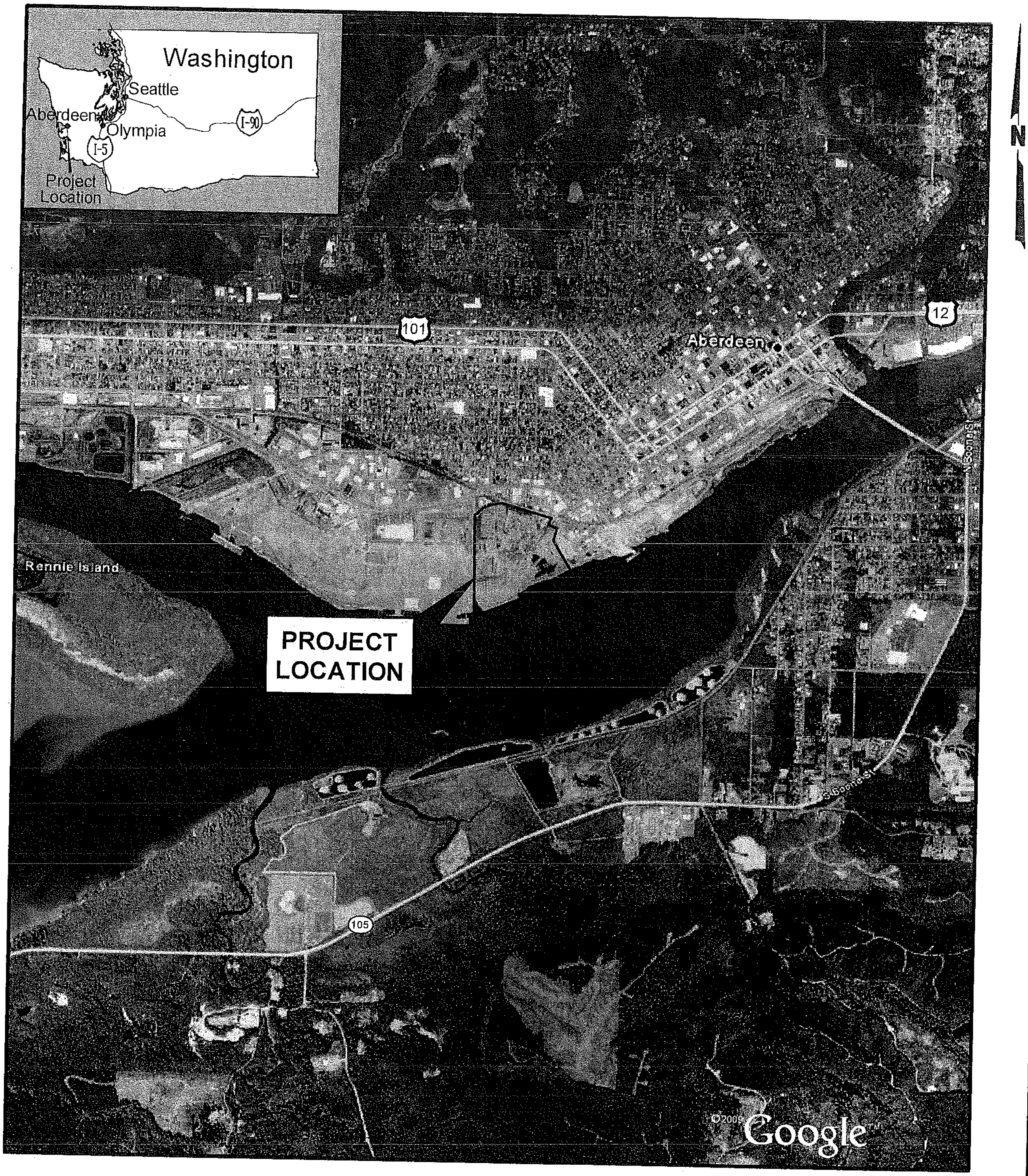
Notes:
1. Passes from a 64-kip axle H300 HD Forklift.
2. The select borrow thickness could be substituted by an equivalent thickness of the existing in situ gravel in these areas.

CBR = California Bearing Ratio

TABLE 8
ASPHALT PAVEMENT SECTIONS

Material	Layer Thickness (inches)			
	Subgrade CBR: 10 Percent		Subgrade CBR: 1 Percent	
	Parking Lot	Entrance Way	Casting Basin Access Road	Entrance Way
HMA	4	5	5	5
Crushed Surfacing Top Course	4	6	6	6
Select Borrow	-	-	30	12
Geogrid and geotextile on the subgrade	No	Yes	Yes	Yes

Notes:
CBR = California Bearing Ratio
HMA = hot mix asphalt



0 3000 6000

Scale in Feet

NOTE

Map adapted from aerial imagery provided by Google Earth Pro, reproduced by permission granted by Google Earth ™ Mapping Service.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

VICINITY MAP

December 2010 21-1-21190-015

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FIG. 1

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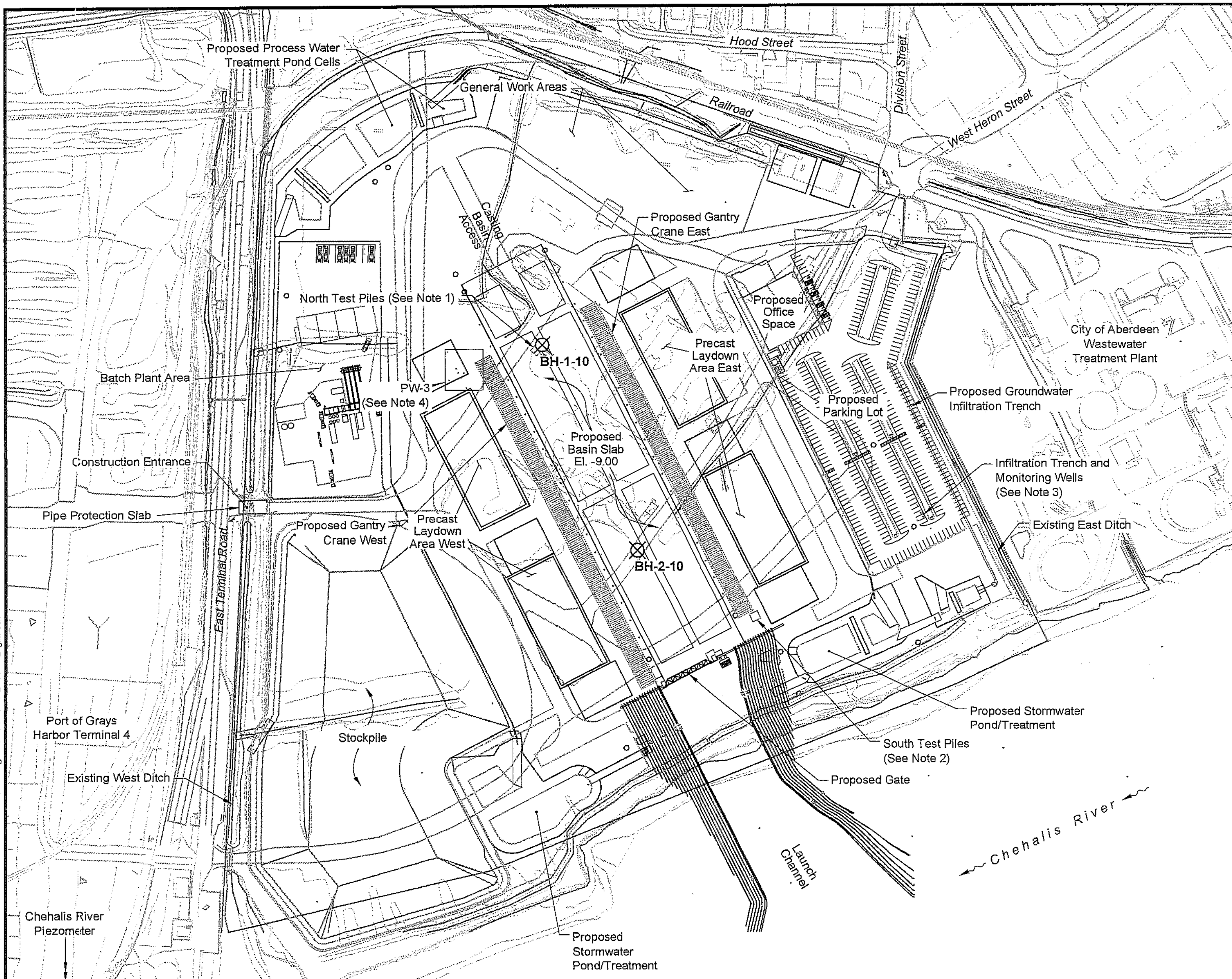

LEGEND

BH-1-10 ⊠ Boring Designation and Approximate Location (S&W 2010)

NOTES

1. North Test Piles: Two 24-inch dia. test piles, two 18-inch dia. test piles, one 20-inch dia. test pile.
2. South Test Piles: Three 24-inch dia. test piles.
3. Infiltration Trench: 12 feet long by 5 feet wide, three 6-inch dia. monitoring wells.
4. Dewatering Wells: Two 24-inch dia. wells, six 6-inch dia. wells.
5. This figure is based on *XL2672_AL_BP_Base Map-Default.dwg* and, *Figure 3-10.dwg*, received 5-10, and *49854_GEO_s_001.dgn*, received 7-26-10.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

SITE AND EXPLORATION PLAN

December 2010 | 21-1-21190-015

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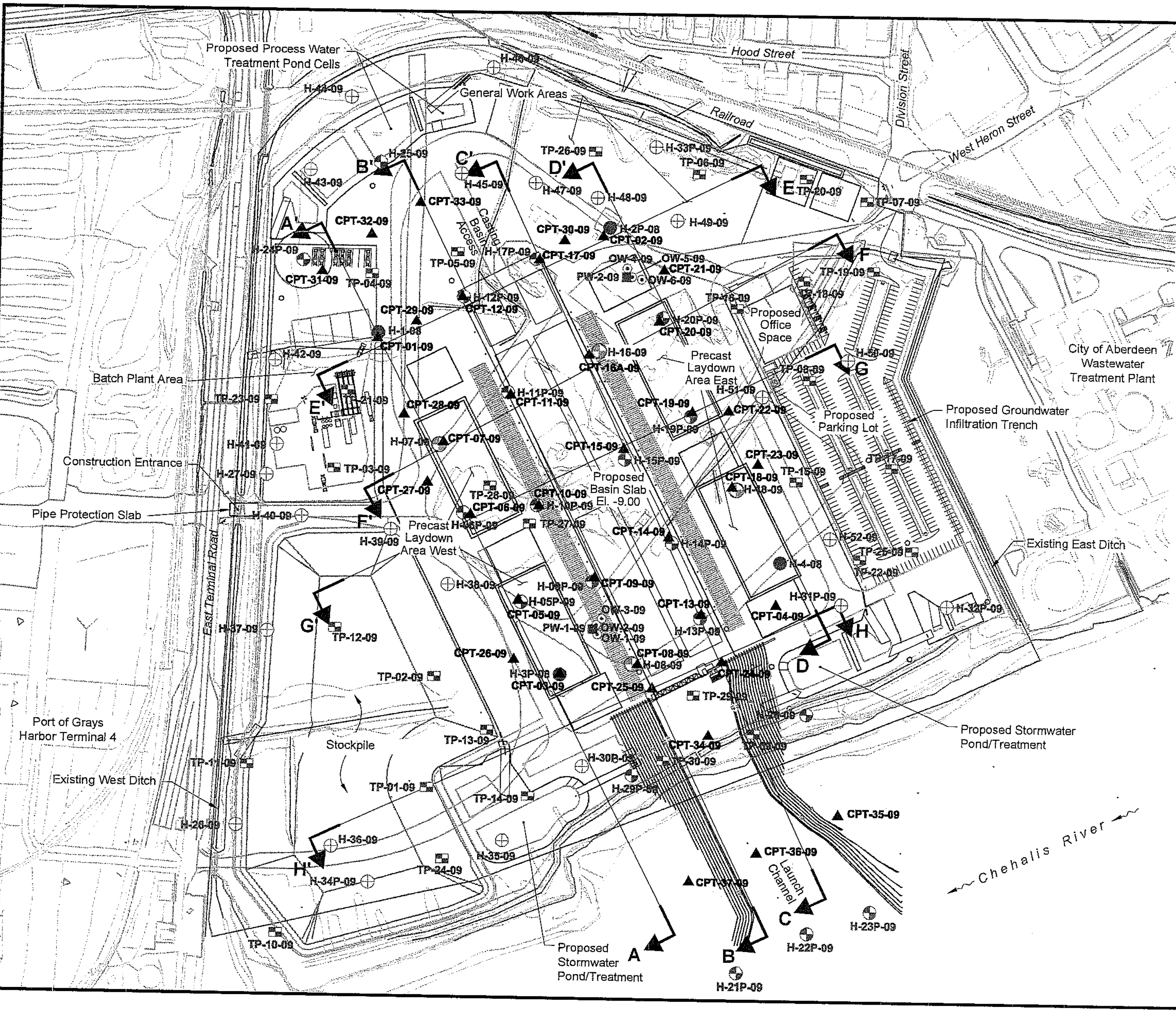
FIG. 2



LEGEND

H-01-08	Borings by Others, Designation and Location, 2008
H-05-09	Borings by Others, Designation and Location, 2009 (Deep Borings - More than 50 Ft)
H-16-09	Borings by Others with Shearwave Logging and Pressuremeter Testing
H-34P-09	Borings by Others, Location and Designation (Shallow Borings - Less than 50 Ft)
	Denotes Piezometer Installation
CPT-01-09	Cone Penetration Tests by Others, Designation and Location
OW-1-09	Observation Wells by Others, Designation and Location
PW-1-09	Pumping Wells by Others, Designation and Location
TP-1-09	Test Pit Designation and Location
A	Generalized Profile of Idealized Soil Behavior Type (See Figures D-1 and D-2)

NOTE

This figure is based on *XL2672_AL_BP_Base Map-Default.dwg* and, *Figure 3-10.dwg*, received 5-10, and *49854_GEO_s_001.dgn*, received 7-26-10.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PREVIOUS SUBSURFACE EXPLORATION PLAN

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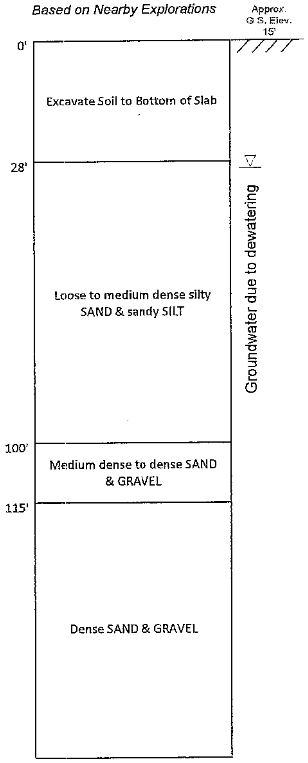
FIG. 3

1/4/2011-18x0.375inch_Closed_North_GDM_DP_Axial_v2.2.xlsm

cij

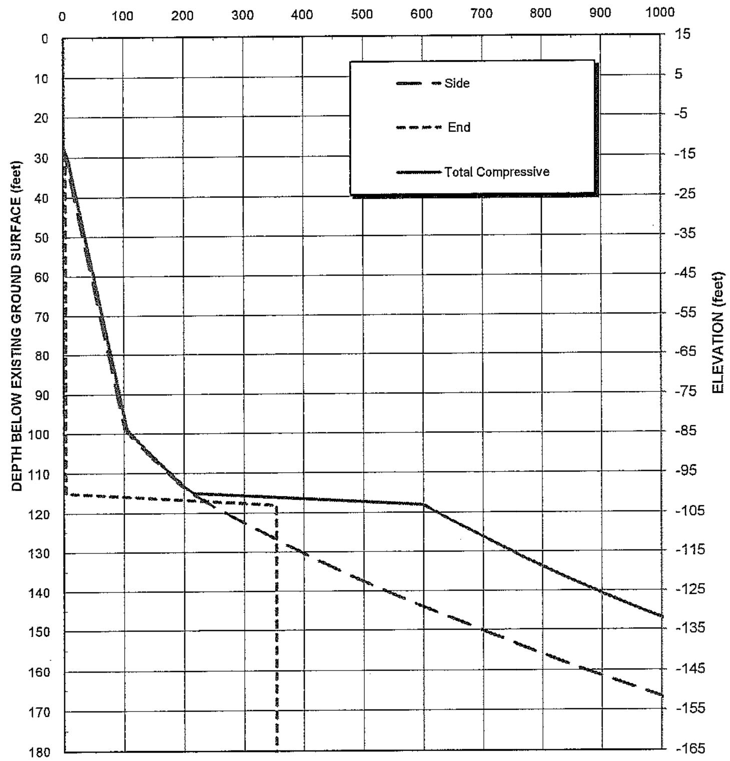
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations



STRENGTH LIMIT

NOMINAL RESISTANCE (kips)



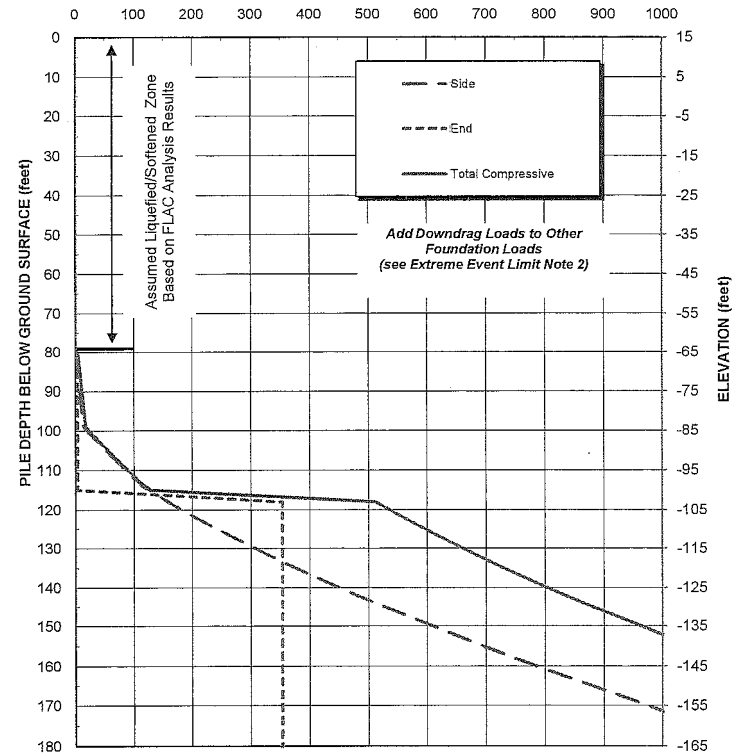
STRENGTH LIMIT NOTES:

1. Recommended resistance factors are 0.65 for side and end resistance.

2. Pile uplift capacity can be estimated by using the unfactored side resistance shown above and a recommended resistance factor of 0.35.

EXTREME EVENT LIMIT

NOMINAL RESISTANCE (kips)



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors are 1.0 for side and base resistance.

2. Unfactored downdrag force is estimated to be 70 kips. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on the guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. Total pile capacity is a summation of its side and end resistances. Nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors. See Strength Limit and Extreme Event Limit Notes above.
3. North profile extends from 350 feet north of the gate structure to the northern extent of the basin.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

ESTIMATED AXIAL RESISTANCE
18-INCH DIA., 3/8-INCH WALL THICK
CLOSED-END PIPE PILE, NORTH

December 2010 | 21-1-21190-015

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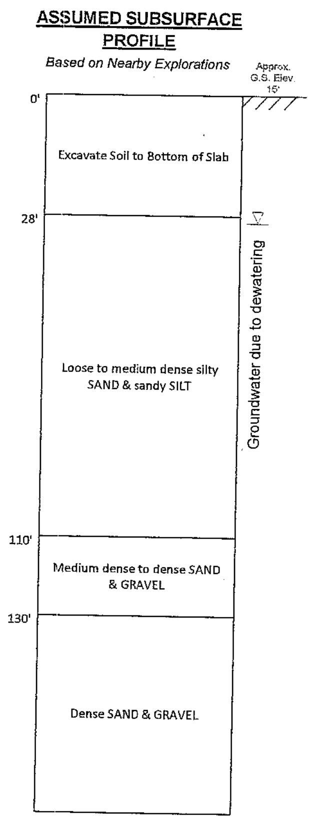
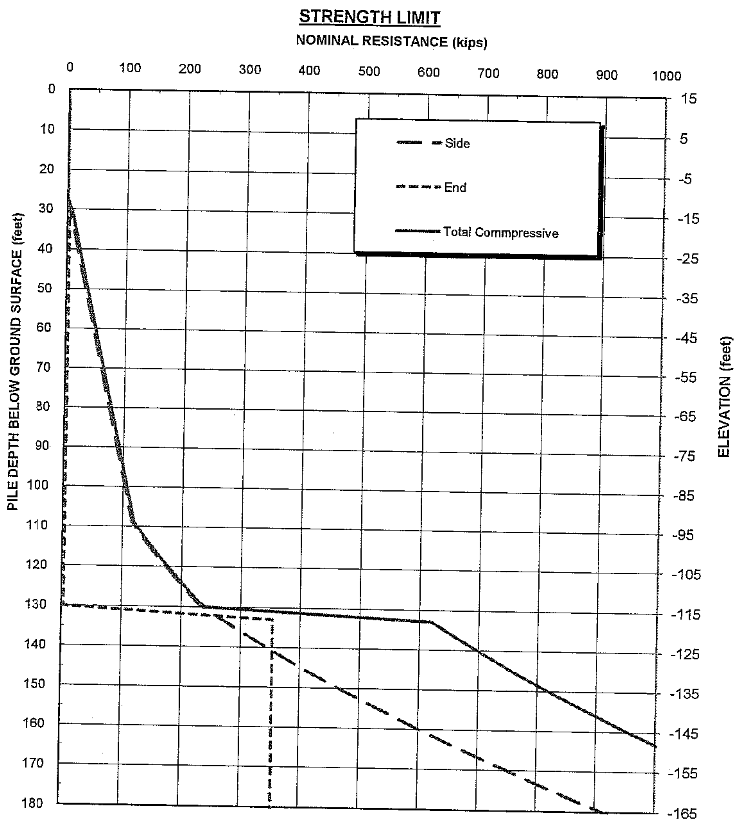
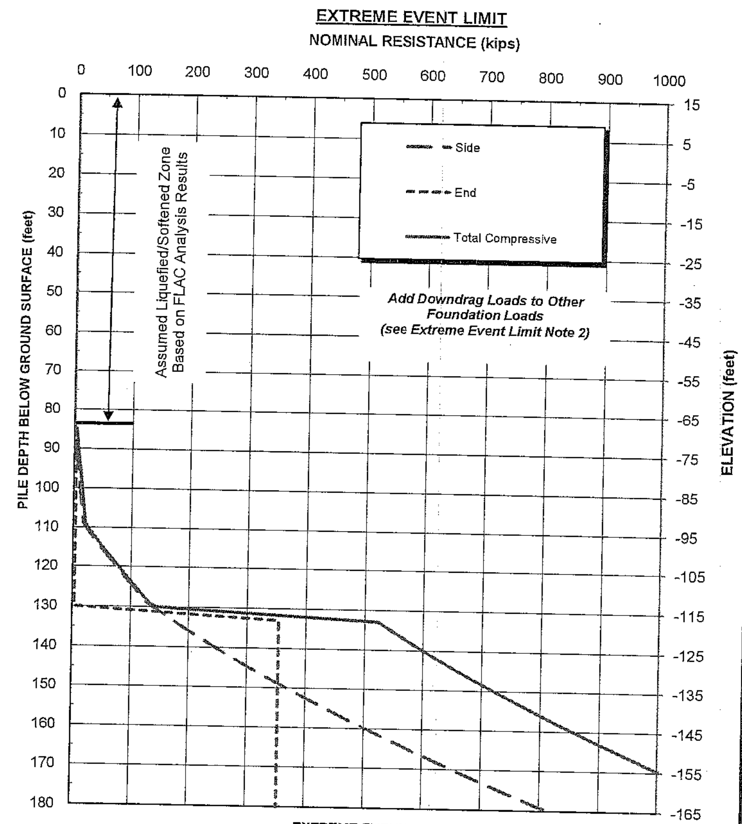
FIG. 4

2/10/2011-18x0.375inch_Closed_South_GDM_DP_Axial_v2.2.xlsm

cij







STRENGTH LIMIT NOTES:

1. Recommended resistance factors are 0.65 for side and end resistance.
2. Pile uplift capacity can be estimated by using the unfactored side resistance shown above and a recommended resistance factor of 0.35.

EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors are 1.0 for side and base resistance.
2. Unfactored downdrag force is estimated to be 40 kips. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on the guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. Total pile capacity is a summation of its side and end resistances. Nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors. See Strength Limit and Extreme Event Limit Notes above.
3. South profile extends from the 200 feet south of the gate structure to 350 feet north of the gate structure.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

ESTIMATED AXIAL RESISTANCE
18-INCH DIA., 3/8-INCH WALL THICK
CLOSED-END PIPE PILE, SOUTH

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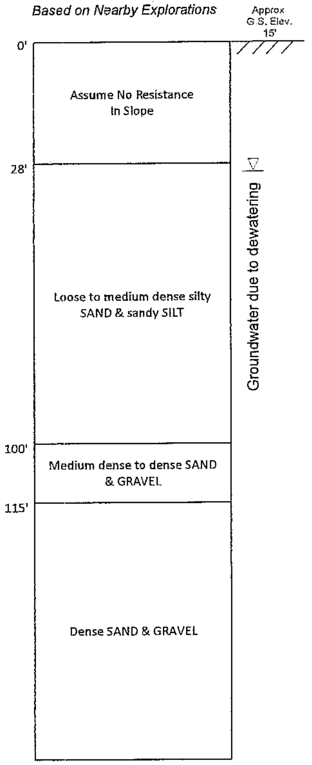
FIG. 5

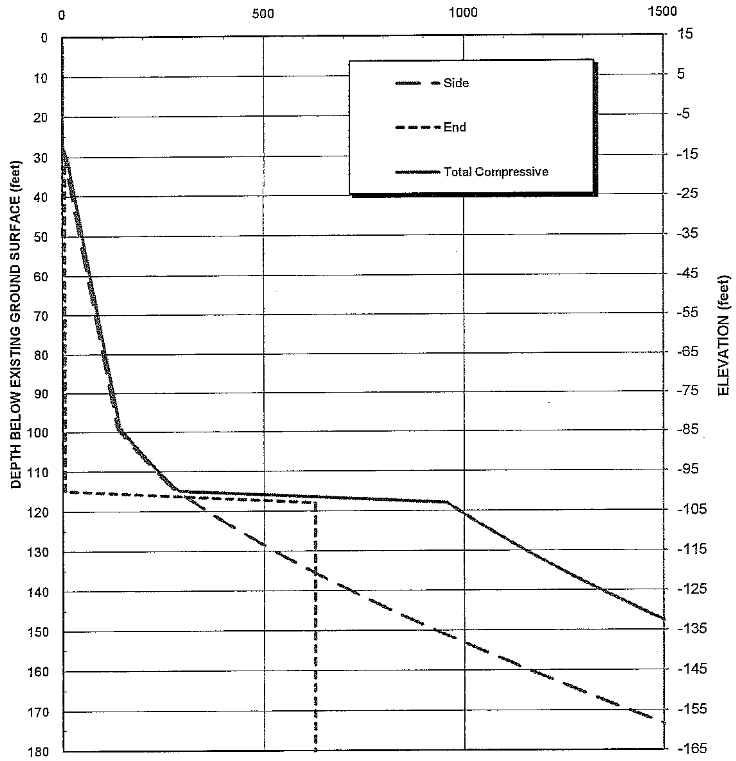
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations



STRENGTH LIMIT

NOMINAL RESISTANCE (kips)

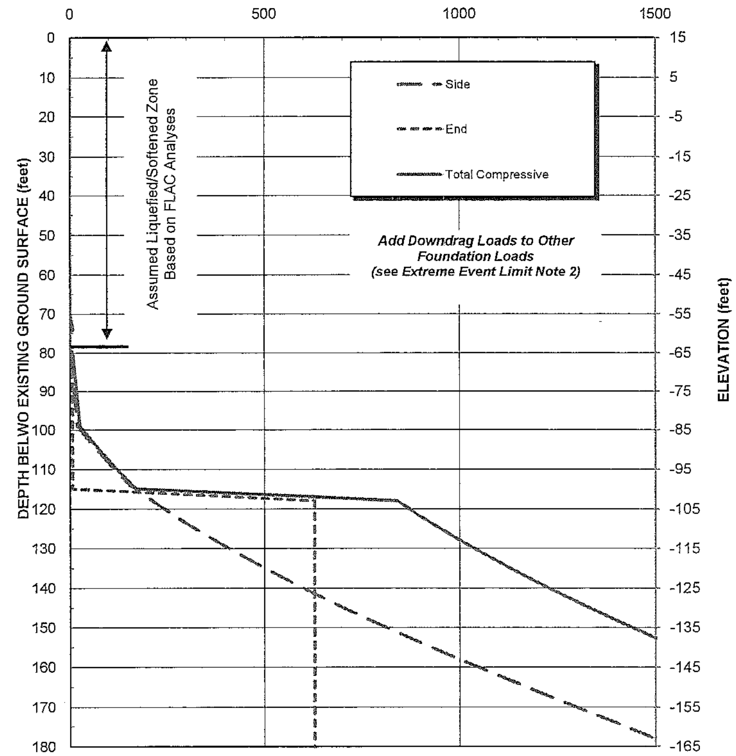


STRENGTH LIMIT NOTES:

1. Recommended resistance factors are 0.65 for side and end resistance.
2. Pile uplift capacity can be estimated by using the unfactored side resistance shown above and a recommended resistance factor of 0.35.

EXTREME EVENT LIMIT

NOMINAL RESISTANCE (kips)



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors are 1.0 for side and base resistance.
2. Unfactored downdrag force is estimated to be 90 kips. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on the guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. Total pile capacity is a summation of its side and end resistances. Nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors. See Strength Limit and Extreme Event Limit Notes above.
3. North profile extends from 350 feet north of the gate structure to the northern extent of the basin.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

**ESTIMATED AXIAL RESISTANCE
24-INCH DIA., 0.401-INCH WALL THICK
CLOSED-END PIPE PILE, NORTH**

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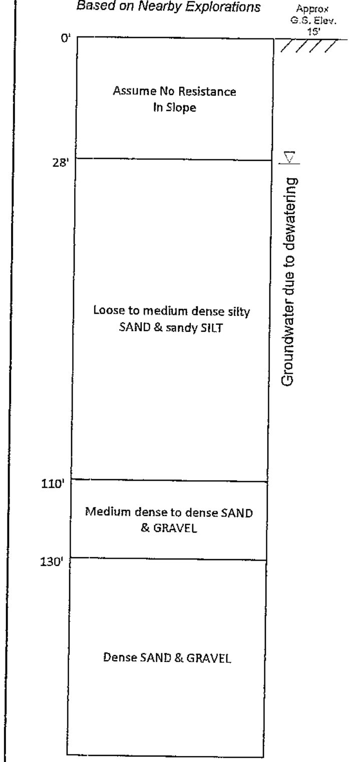
FIG. 6

2/10/2011-24x0.401inch_Closed_South_GDM_DP_Axial_v2.2.xlsm

cij

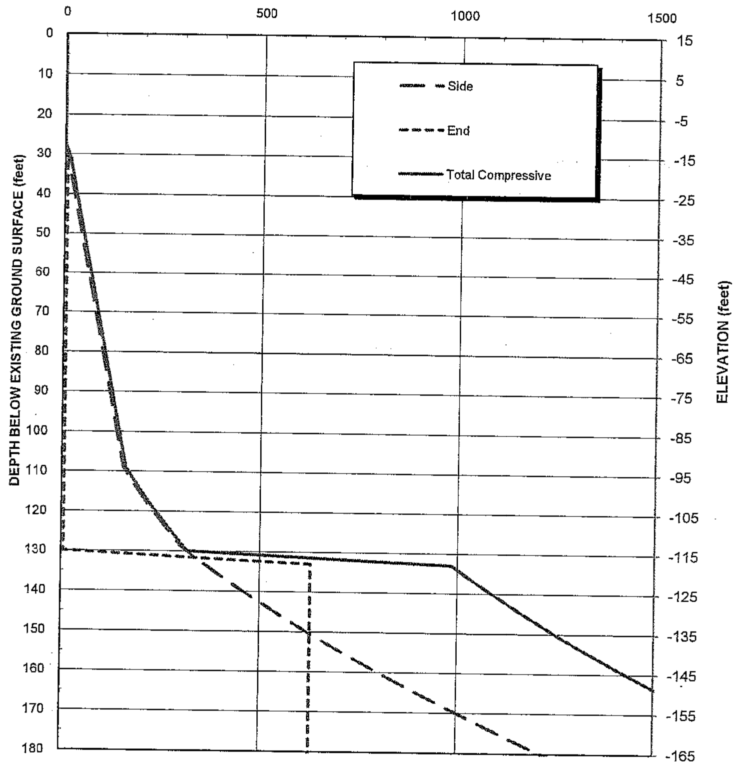
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations



STRENGTH LIMIT

NOMINAL RESISTANCE (kips)

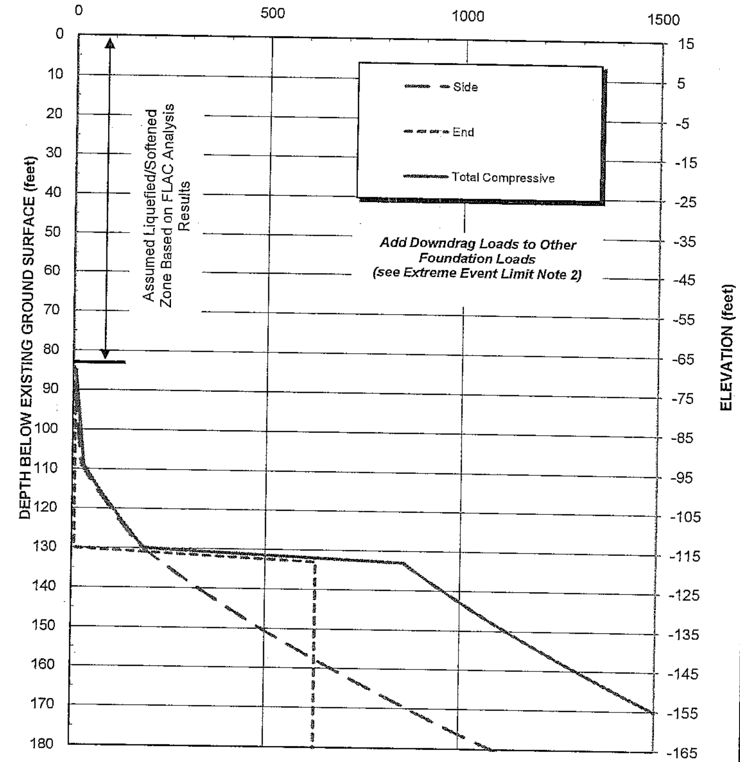


STRENGTH LIMIT NOTES:

1. Recommended resistance factors are 0.65 for side and end resistance.
2. Pile uplift capacity can be estimated by using the unfactored side resistance shown above and a recommended resistance factor of 0.35.

EXTREME EVENT LIMIT

NOMINAL RESISTANCE (kips)



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors are 1.0 for side and base resistance.
2. Unfactored downdrag force is estimated to be 60 kips. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on the guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. Total pile capacity is a summation of its side and end resistances. Nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors. See Strength Limit and Extreme Event Limit Notes above.
3. South profile extends from the 200 feet south of the gate structure to 350 feet north of the gate structure.

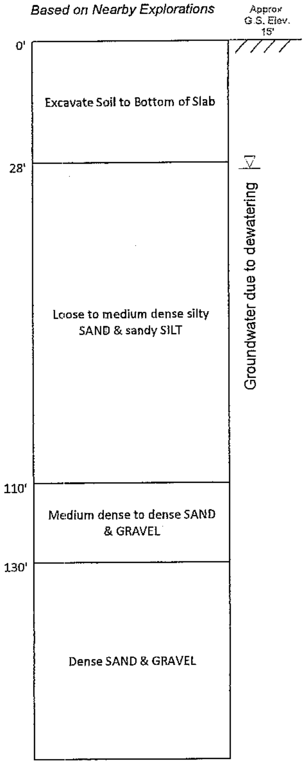
SR 520 Pontoon Casting Facility Aberdeen, Washington	
ESTIMATED AXIAL RESISTANCE 24-INCH DIA., 0.401-INCH WALL THICK CLOSED-END PIPE PILE, SOUTH	
December 2010	21-1-21190-015
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. 7

2/10/2011-24x0.401inch_Open_South_GDM_DP_Axial_v2.2.xlsm

cij

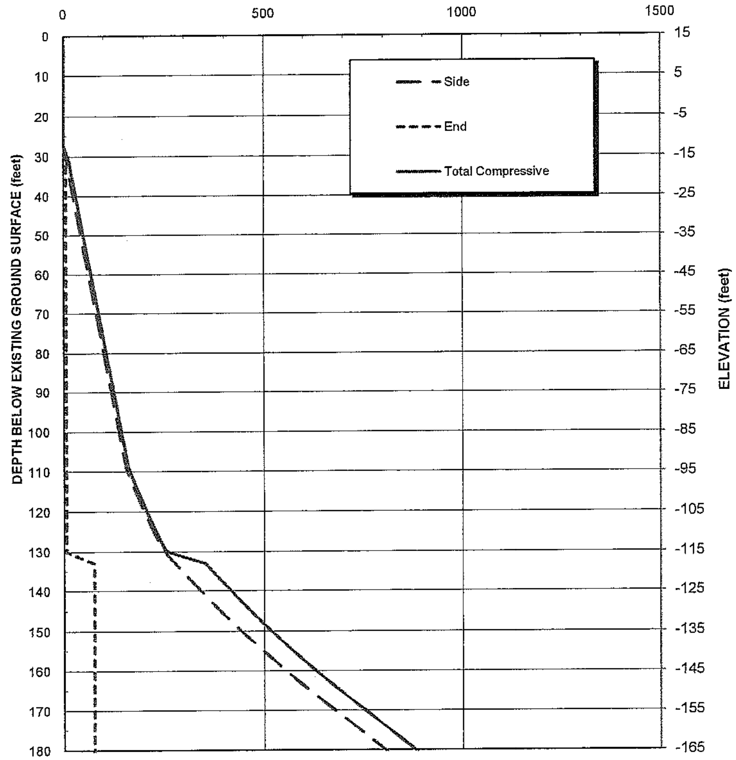
ASSUMED SUBSURFACE PROFILE

Based on Nearby Explorations



STRENGTH LIMIT

NOMINAL RESISTANCE (kips)

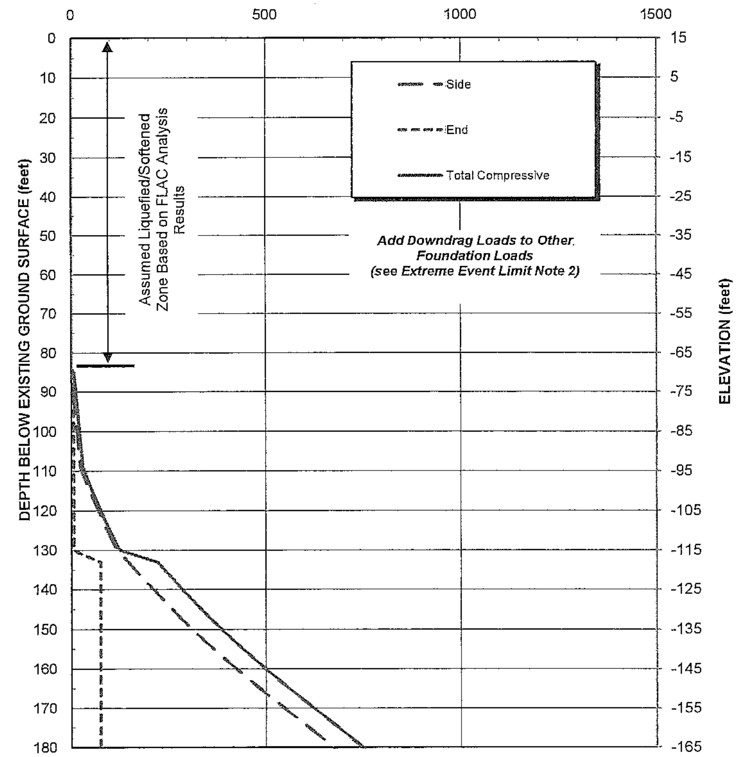


STRENGTH LIMIT NOTES:

1. Recommended resistance factors are 0.65 for side and end resistance.
2. Pile uplift capacity can be estimated by using the unfactored side resistance shown above and a recommended resistance factor of 0.35.

EXTREME EVENT LIMIT

NOMINAL RESISTANCE (kips)



EXTREME EVENT LIMIT NOTES:

1. Recommended resistance factors are 1.0 for side and end resistance.
2. Unfactored downdrag force is estimated to be 60 kips. Per the WSDOT GDM, a load factor of 1.25 is recommended to determine factored downdrag force. Downdrag force is recommended to be applied with post-earthquake loading.

GENERAL NOTES

1. The analyses were performed based on the guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. Total pile capacity is a summation of its side and end resistances. Nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors. See Strength Limit and Extreme Event Limit Notes above.
3. South profile extends from the 200 feet south of the gate structure to 350 feet north of the gate structure.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

ESTIMATED AXIAL RESISTANCE
24-INCH DIA., 0.401-INCH WALL THICK
OPEN-END PIPE PILE, SOUTH

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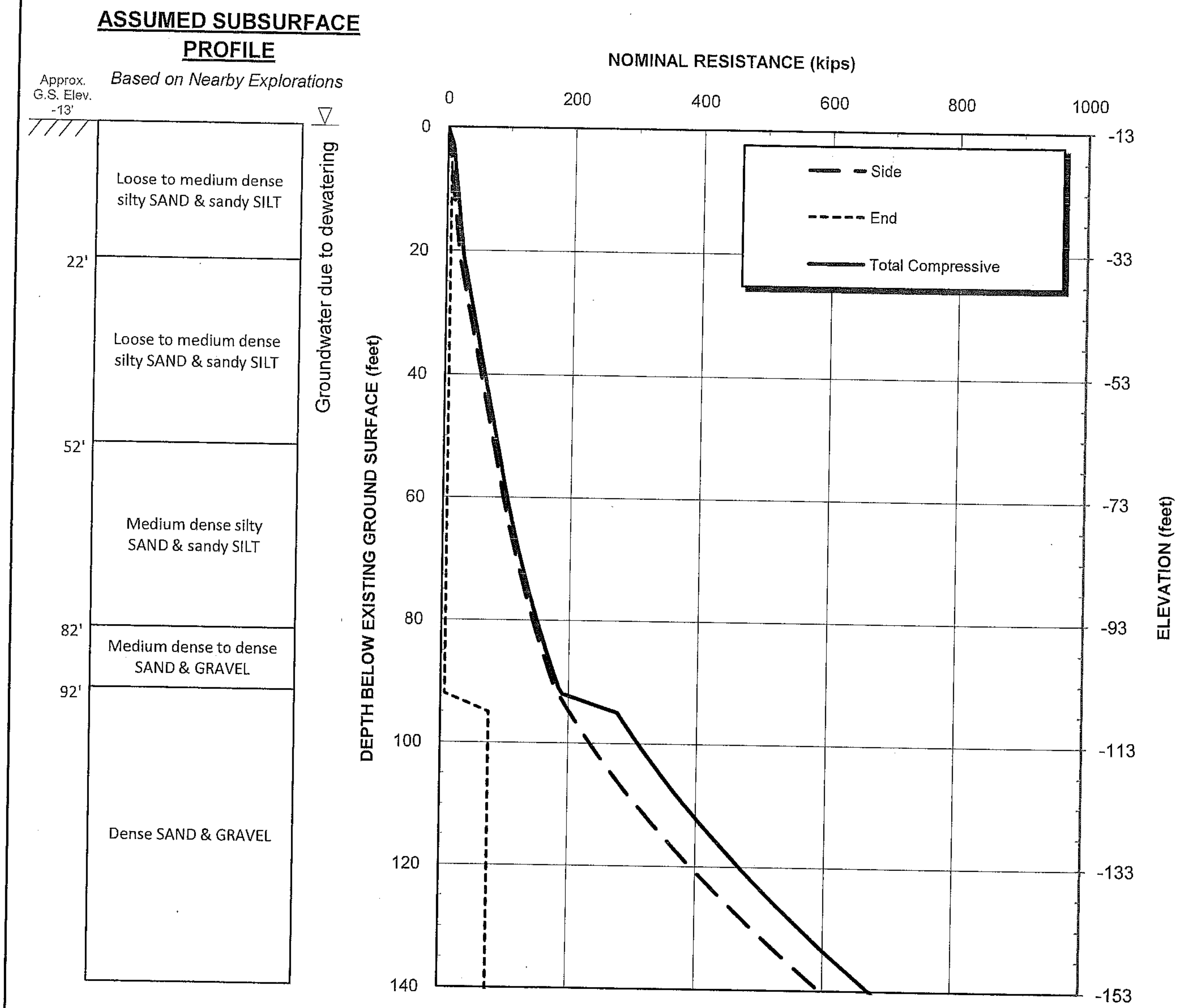
FIG. 8



NOTES:

1. The analyses were performed based on the guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. Total pile capacity is a summation of its side and end resistances. Nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors. See note 3 for recommended resistance factors.
3. For the strength limit state, recommended resistance factors (RF) are:
   RF = 0.65 for both side and end resistance and
   RF = 0.35 for uplift resistance.
4. The downdgrag load was not estimated for the Dolphin piles.
5. Offshore profile extends from the 200 feet south of the gate structure to the southern extent of the project site.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

ESTIMATED AXIAL RESISTANCE
24-INCH DIA., 0.401-INCH WALL THICK
OPEN-END PIPE PILE, OFFSHORE

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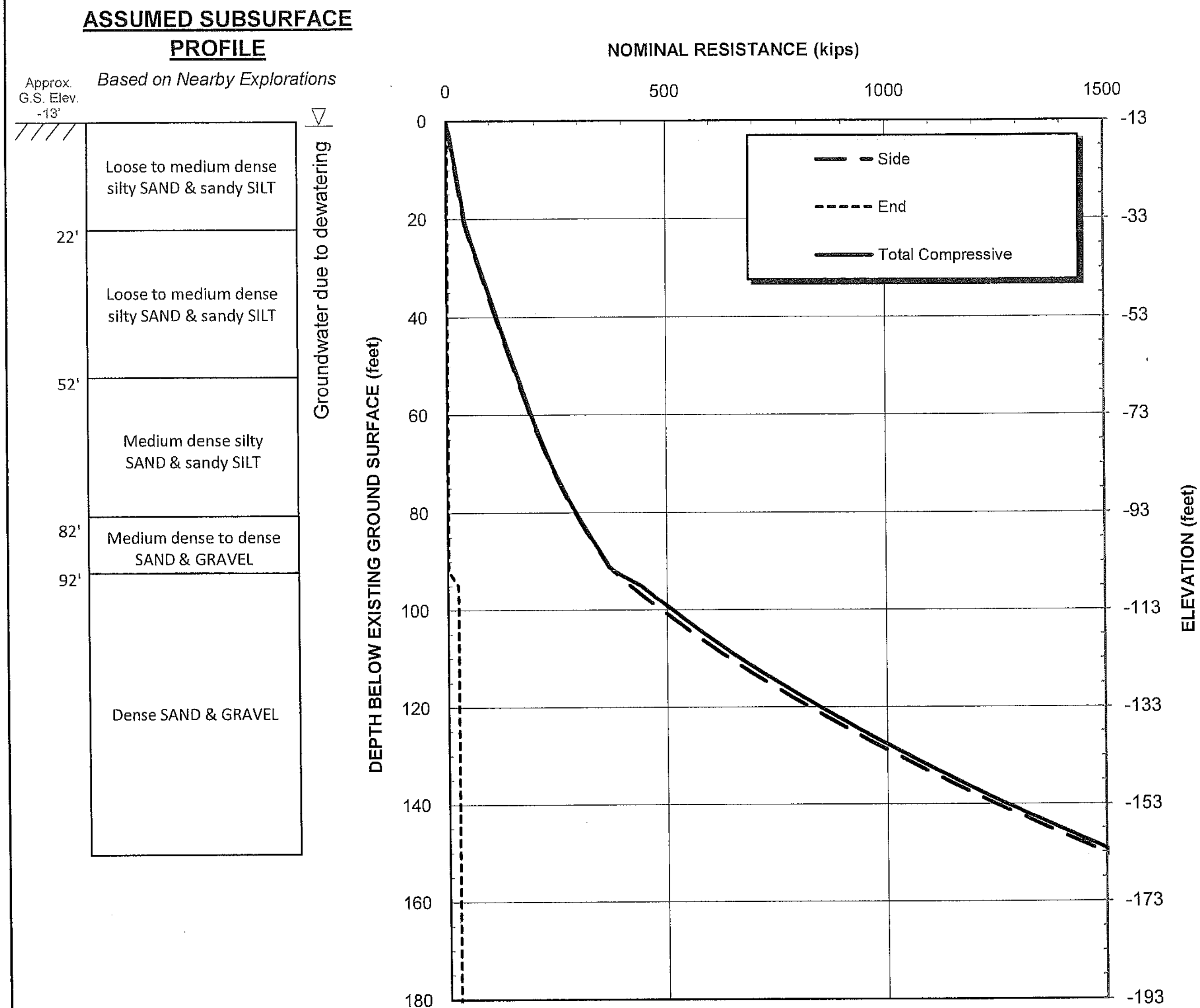
FIG. 9



NOTES:

1. The analyses were performed based on the guidelines included in the WSDOT Geotechnical Design Manual (GDM) and local experience. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).

2. Total pile capacity is a summation of its side and end resistances. Nominal resistances shown on plots above are to be multiplied by the appropriate resistance factors. See note 3 for recommended resistance factors.

3. For the strength limit state, recommended resistance factors (RF) are:
   RF = 0.65 for both side and end resistance and
   RF = 0.35 for uplift resistance.

4. The downdgrag load was not estimated for the Dolphin piles.

5. Offshore profile extends from the 200 feet south of the gate structure to the southern extent of the project site.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

ESTIMATED AXIAL RESISTANCE
48-INCH DIA., 1-INCH WALL THICK
OPEN-END PIPE PILE, OFFSHORE

December 2010 21-1-21190-015

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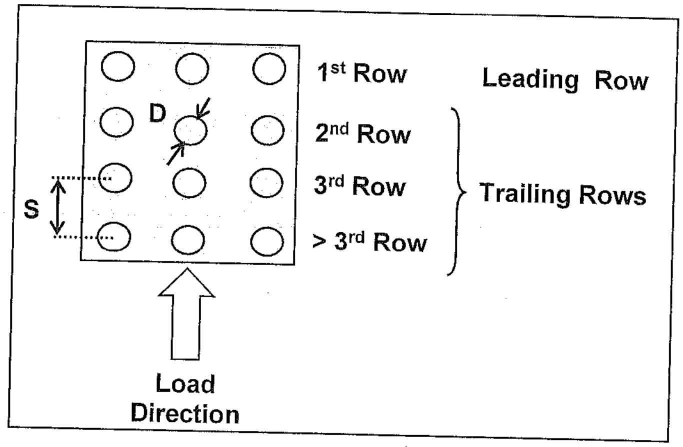
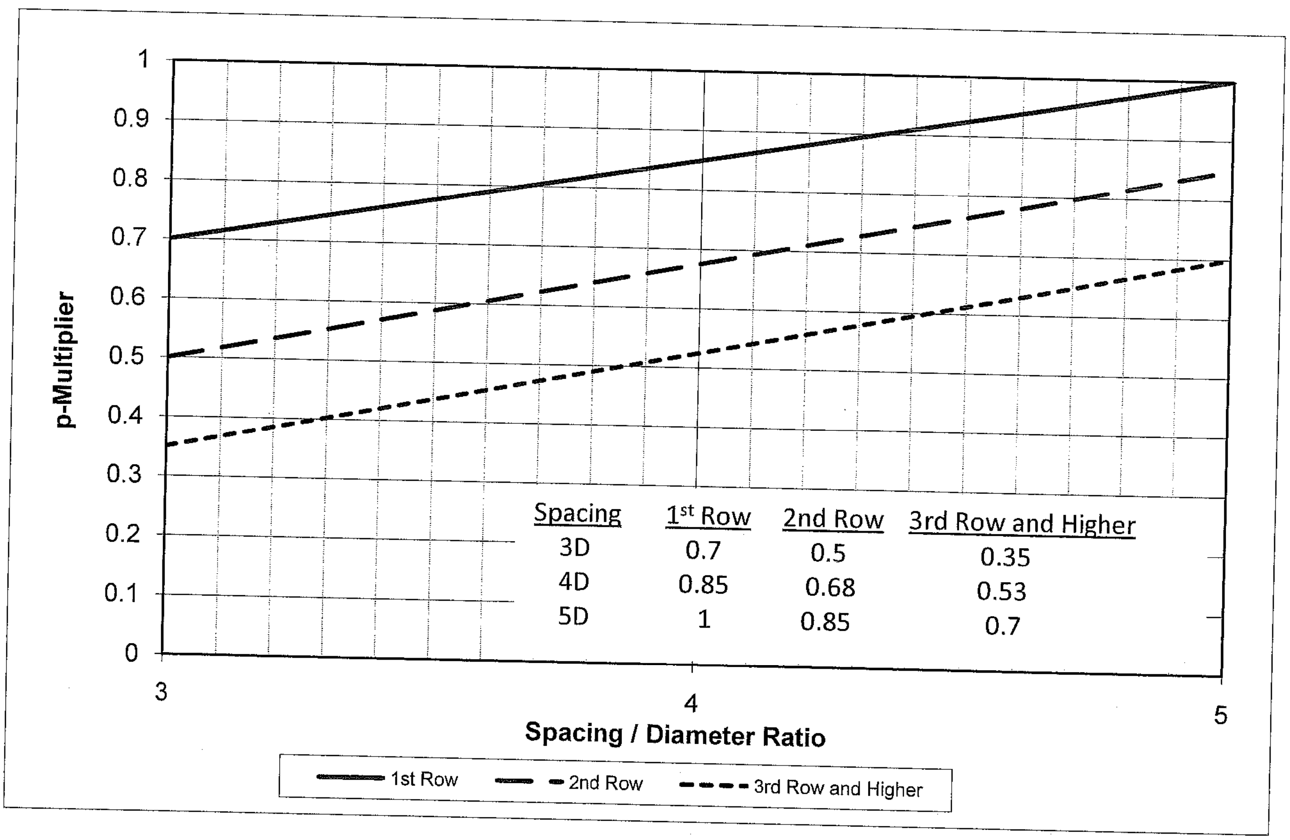
FIG. 10





Spacing	1st Row	2nd Row	3rd Row and Higher
3D	0.7	0.5	0.35
4D	0.85	0.68	0.53
5D	1	0.85	0.7

NOTES

1. Developed in general accordance with Section 10.7.2.4 AASHTO LRFD Design Manual (2008 Interim).
2. S = Center-to-center pile spacing
   D = Pile Diameter

SR 520 Pontoon Casting Facility
Aberdeen, Washington

P-MULTIPLIER
GROUP EFFECT OF LATERALLY LOADED PILES

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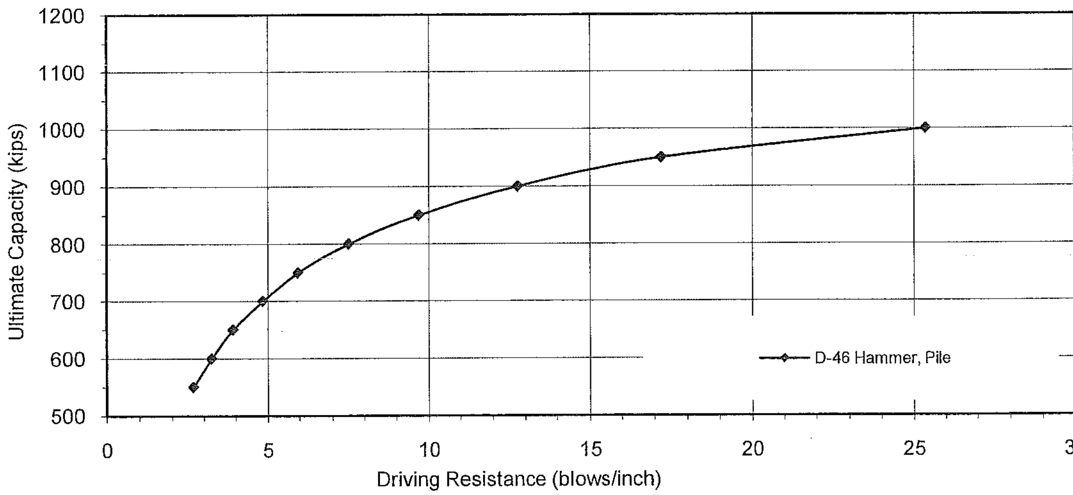
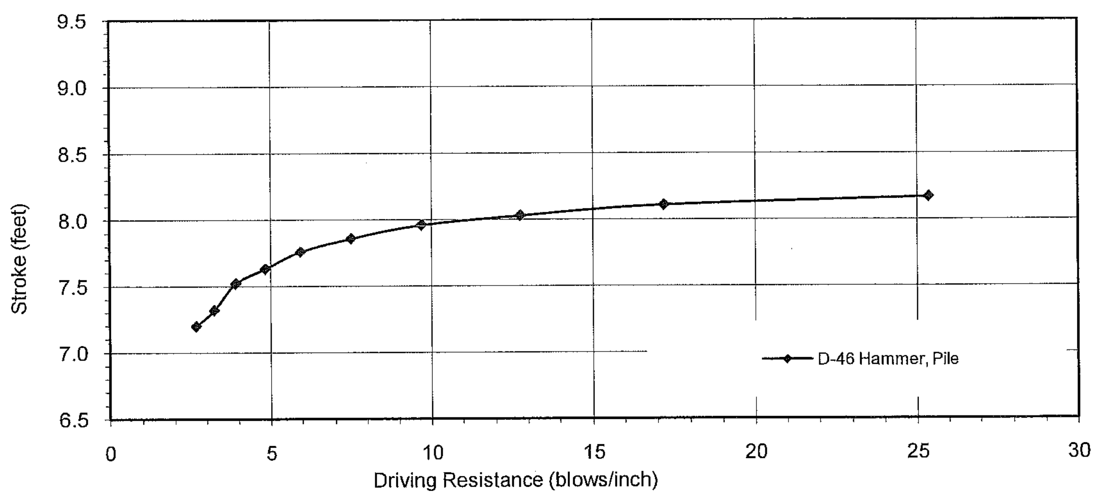
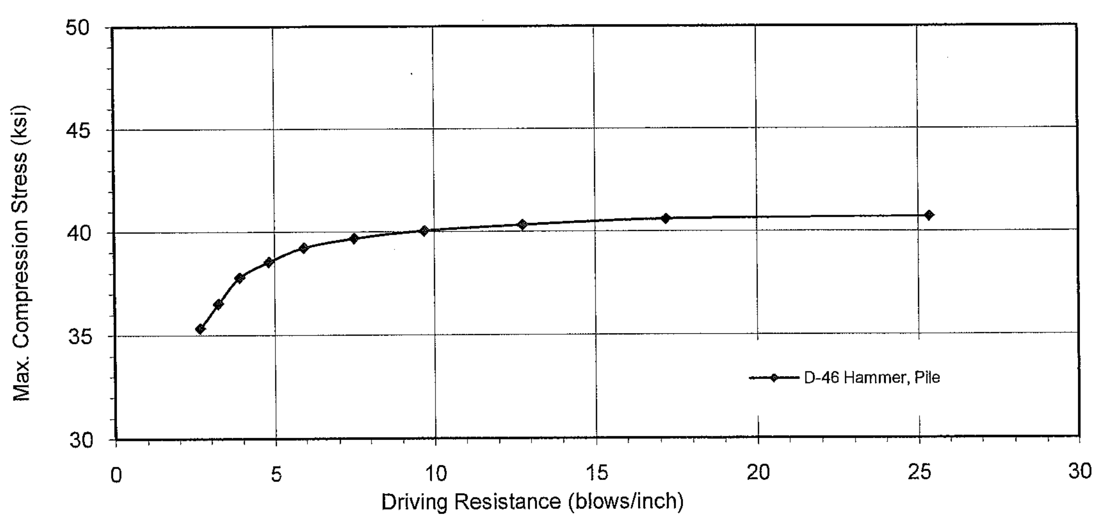
FIG. 11







Notes:

1. The computer program GRLWEAP (PDI, 1998) was used for the WEAP analyses.
2. GRLWEAP recommended values used for quake, damping and helmet parameters.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

WEAP ANALYSIS
18-INCH-DIA., 3/8-INCH WALL THICK CLOSED-END PIPE PILE

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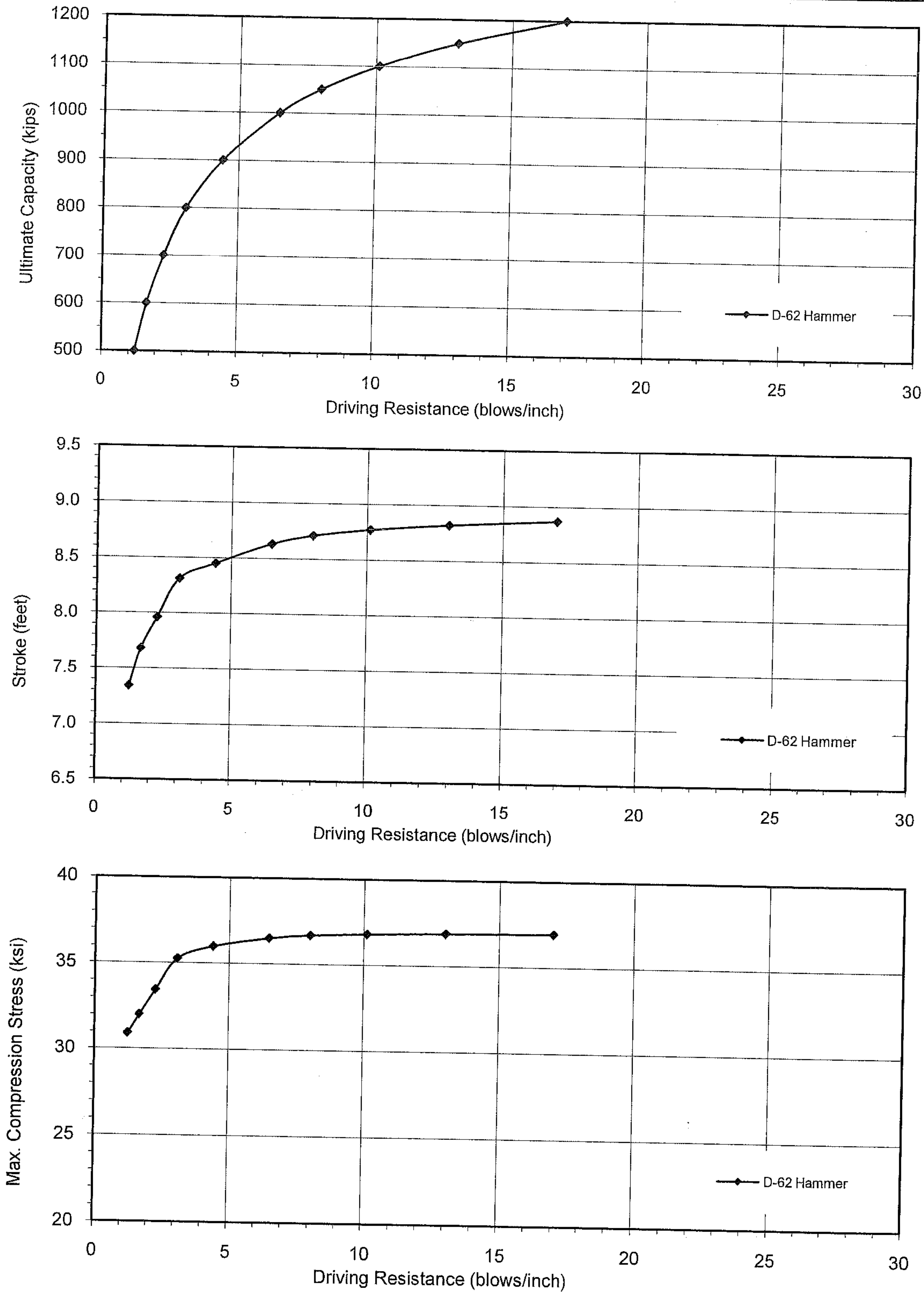
FIG. 12



Notes:

1. The computer program GRLWEAP (PDI, 1998) was used for the WEAP analyses.
2. GRLWEAP recommended values used for quake, damping and helmet parameters.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

WEAP ANALYSIS
24-INCH-DIA., 0.401-INCH WALL THICK
CLOSED-END PIPE PILE

December 2010 21-1-21190-015

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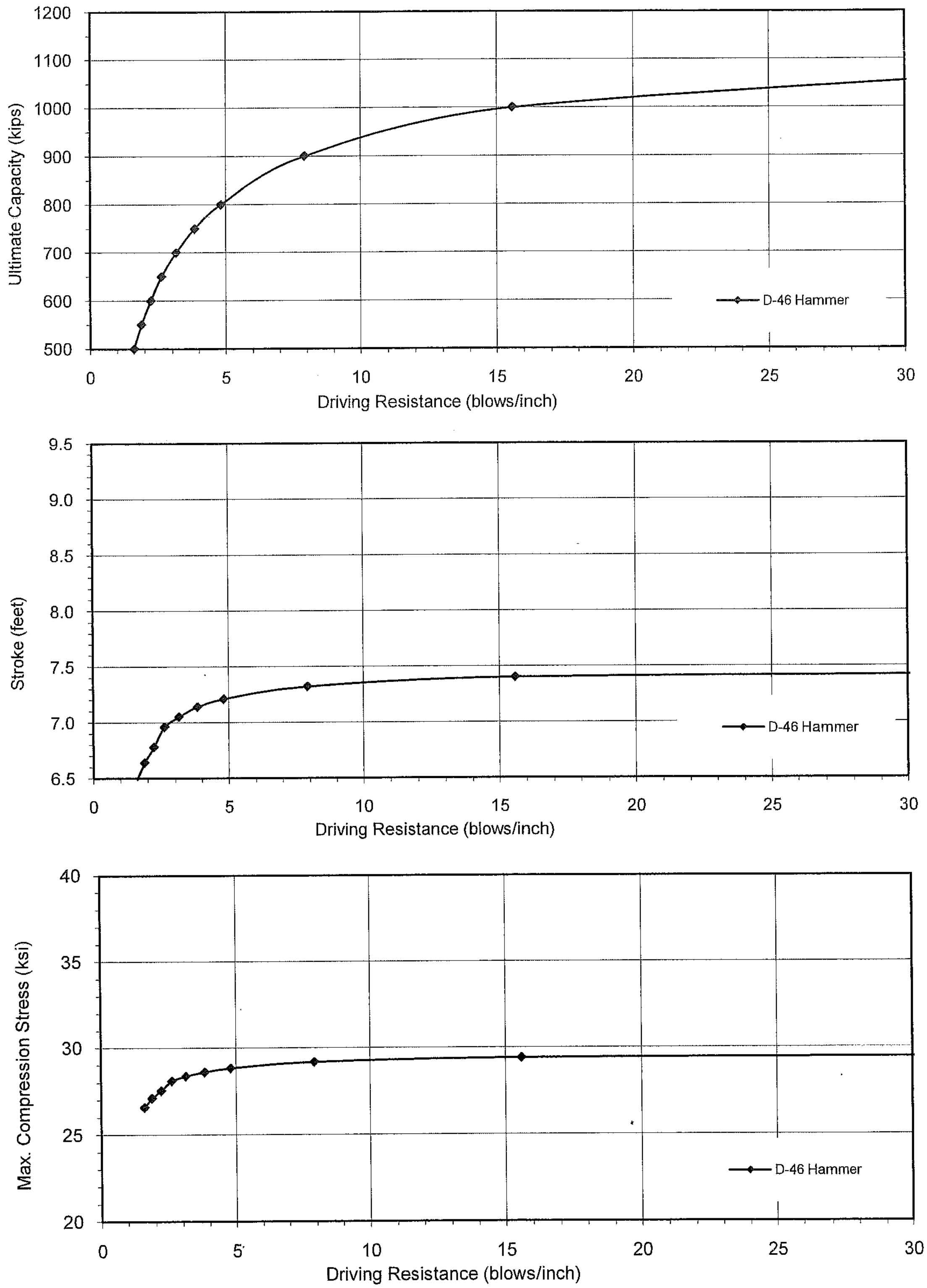
FIG. 13



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Aberdeen, Washington

WEAP ANALYSIS
24-INCH-DIA., 0.401-INCH WALL THICK
OPEN-END PIPE PILE

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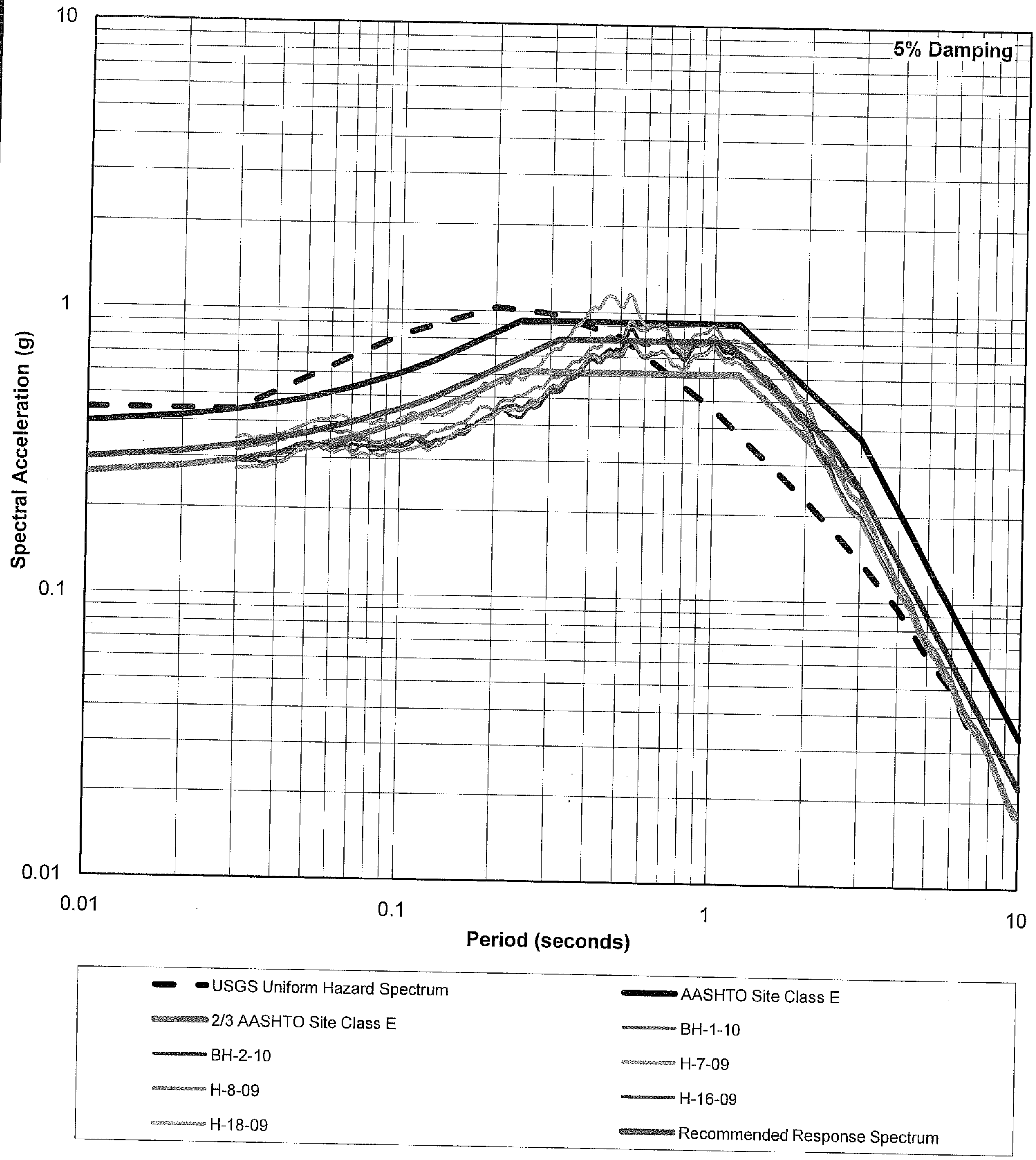
FIG. 14

Notes:

1. The computer program GRLWEAP (PDI, 1998) was used for the WEAP analyses.
2. GRLWEAP recommended values used for quake, damping and helmet parameters.

jnb



NOTES

1. The plot for each boring is the geometric mean of the total stress response of the seven time histories.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

RECOMMENDED ACCELERATION RESPONSE SPECTRUM

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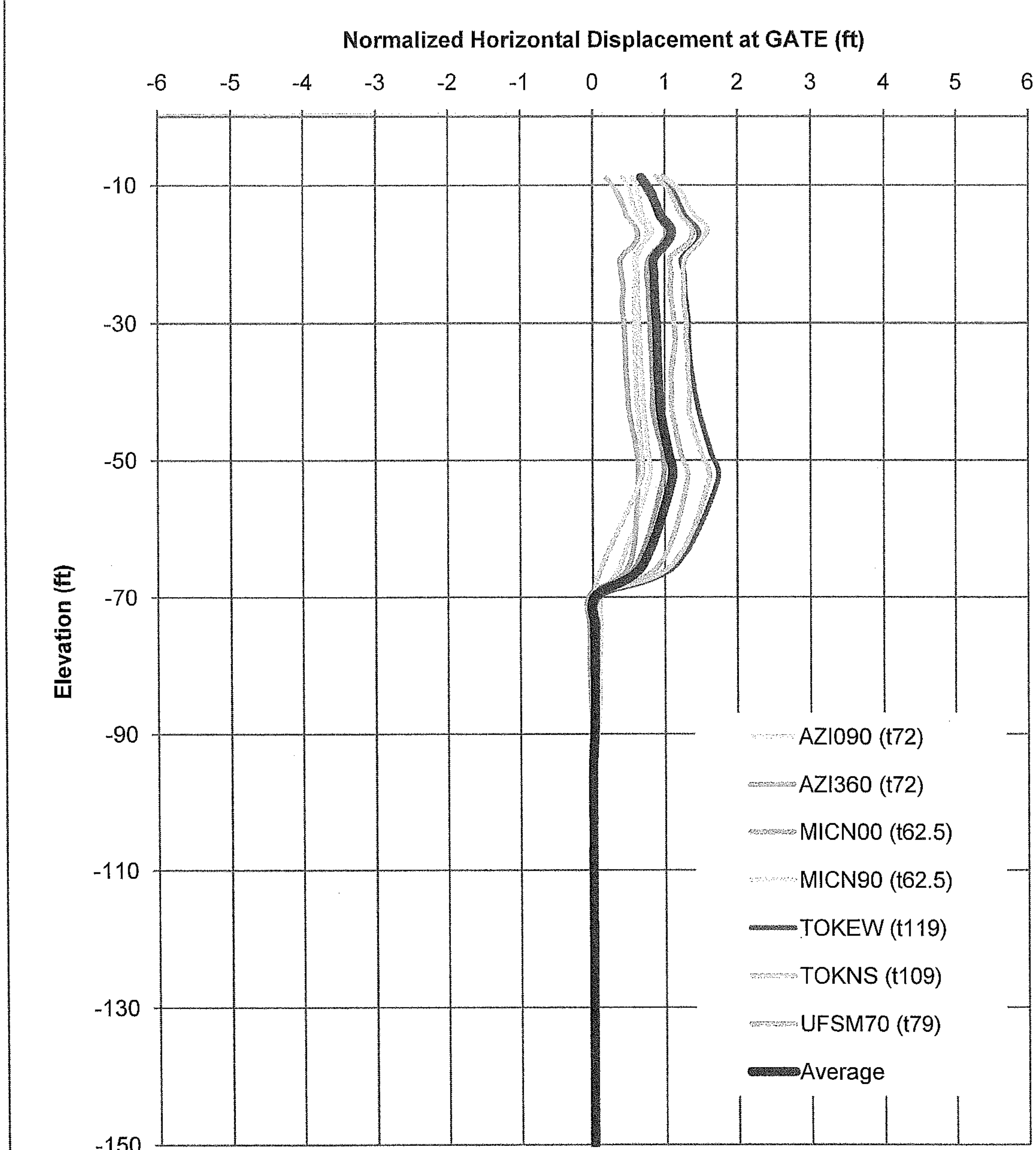
FIG. 15



NOTES

1. Horizontal displacements do not include soil-structure interaction effects and are considered "Free Field"

SR 520 Pontoon Casting Facility
Aberdeen, Washington

LONGITUDINAL - CENTER BASIN FREE-FIELD HORIZONTAL GROUND DISPLACEMENTS

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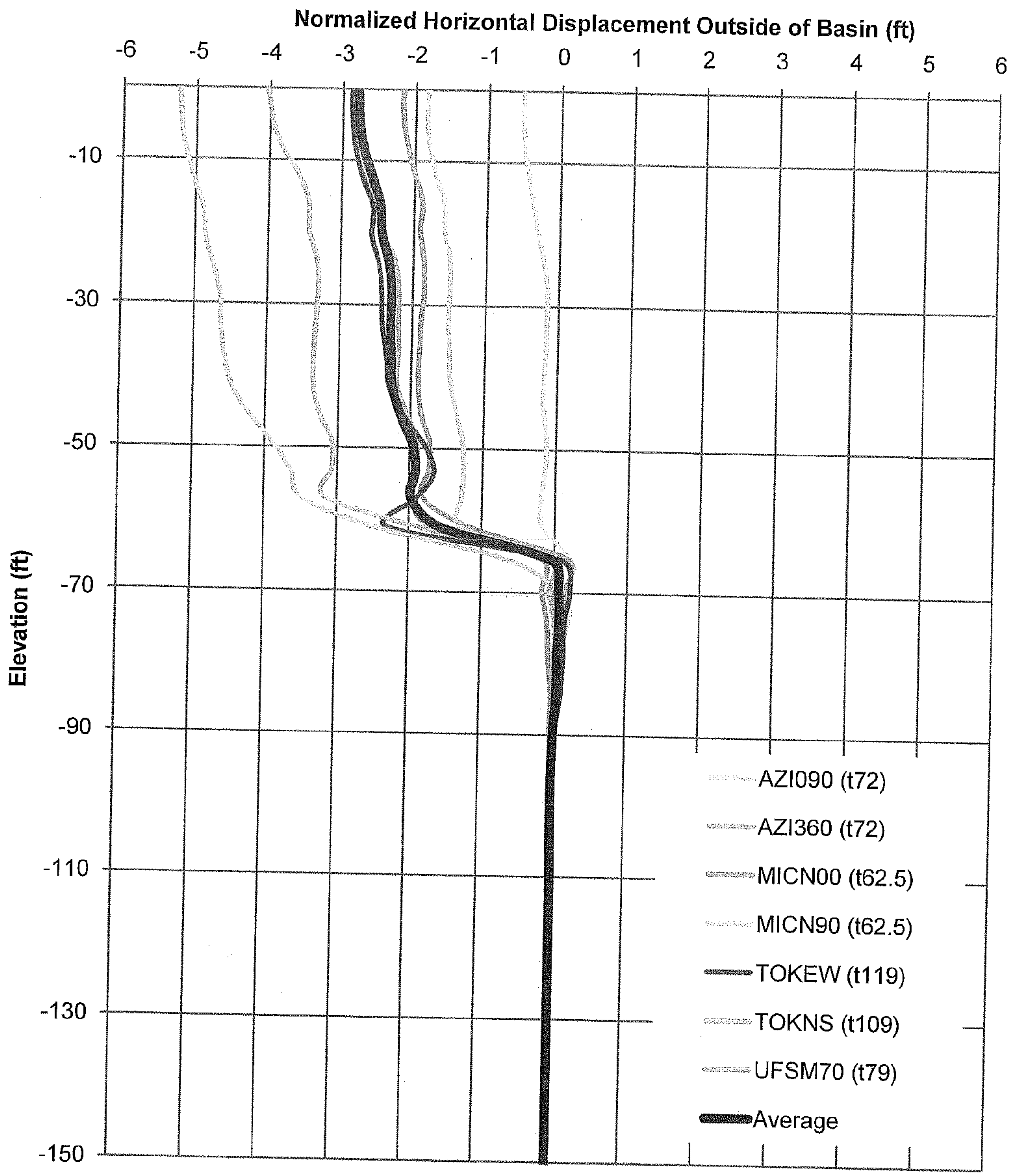
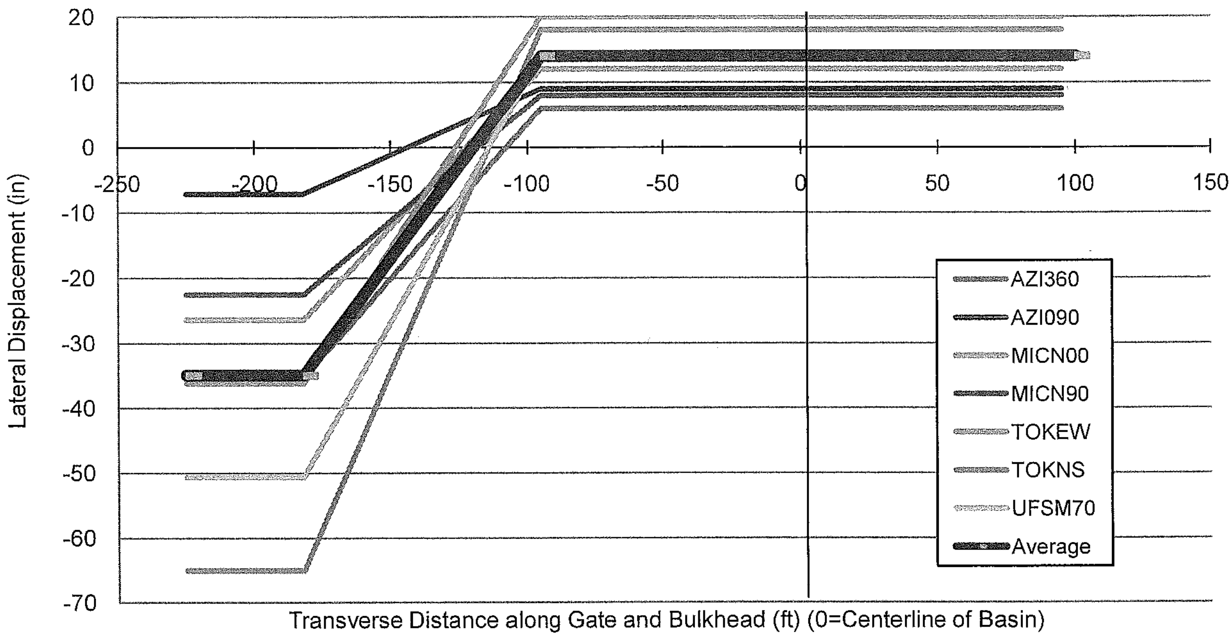
FIG. 16



NOTES

1. Horizontal displacements do not include soil-structure interaction effects and should be considered "Free Field"

SR 520 Pontoon Casting Facility Aberdeen, Washington	
LONGITUDINAL - OUTSIDE BASIN FREE-FIELD HORIZONTAL GROUND DISPLACEMENTS	
December 2010	21-1-21190-016
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 17**



Notes

1. Positive displacement is north.

2. Analyses were performed in the bottom of the basin and at the top of the slope. The interpolation shown above between the results of the two analyses was performed to assist the structural engineer.

SR520 Pontoon Casting Facility
Aberdeen, Washington

GATE
FREE-FIELD GROUND SURFACE
SOIL DISPLACEMENT

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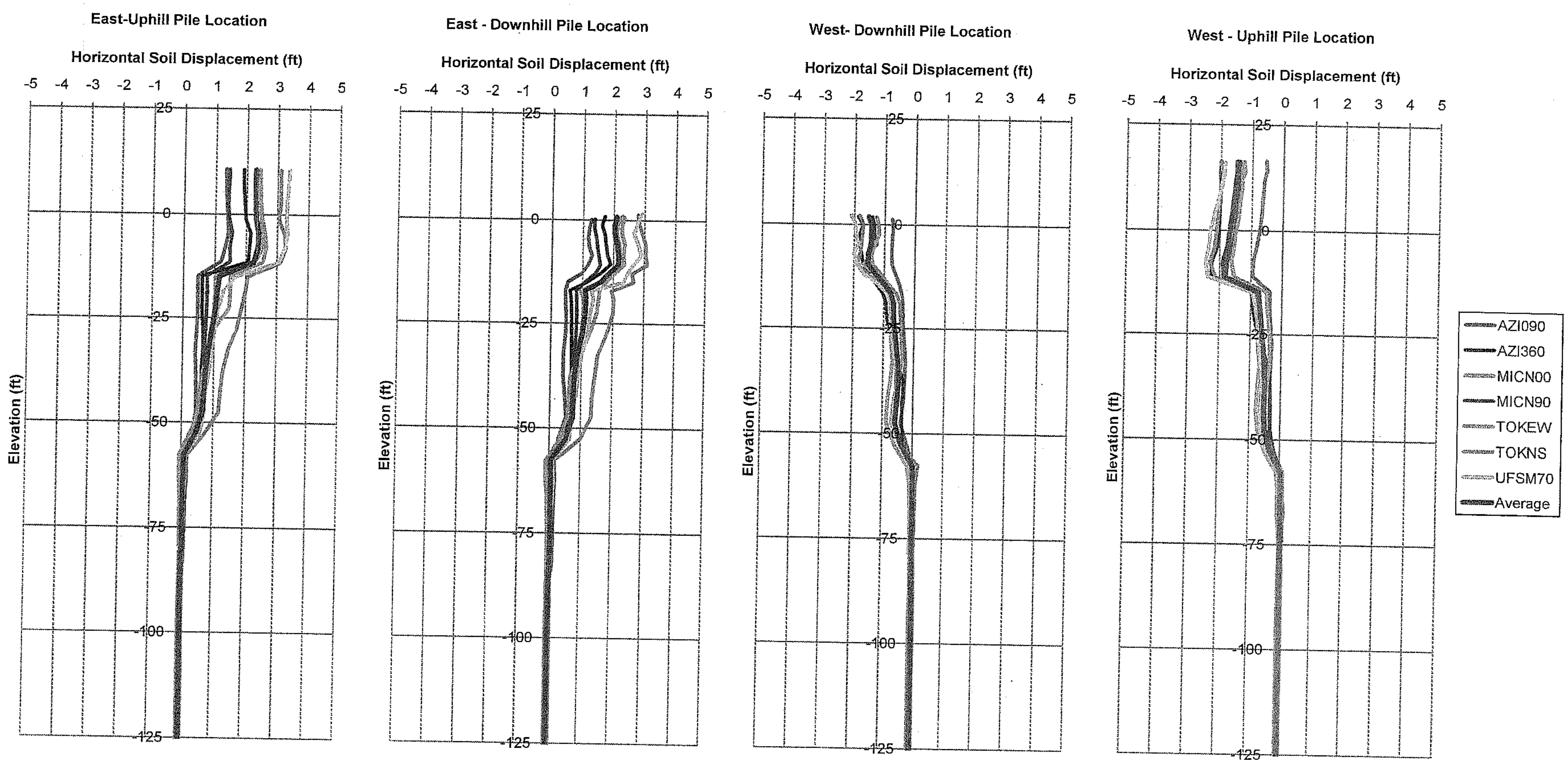
FIG. 18



NOTES

1. Ground displacements are near crane trestle pile loactions.
2. Results represent the effects of soil-structure interaction.
3. Results are taken at the end of ground motion shaking.
4. Positive displacments represent movements to the west and negative displacements represent movements to the east.

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Aberdeen, Washington

TRANSVERSE - NORTH BASIN
HORIZ. SOIL DISPLACEMENT

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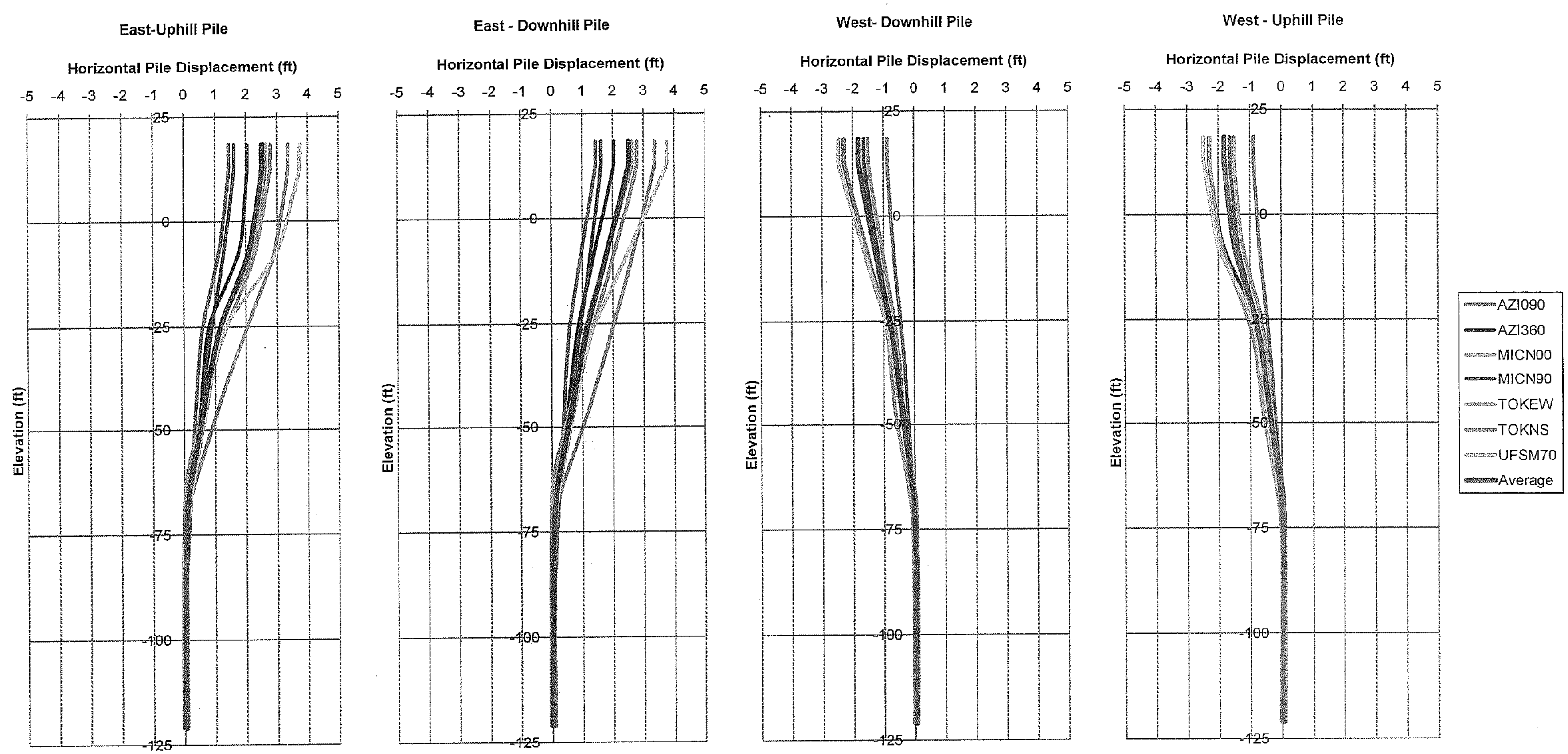
FIG. 19

Figures (North).xlsm Printed: 12/30/2010 4:20 PM



NOTES

1. Pile response includes plastic hinging behavior. See text for plastic yield moments.
2. Results represent the effects of soil-structure interaction.
3. Results are taken at the end of ground motion shaking.
4. Positive displacments represent movements to the west and negative displacements represent movements to the east.

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Aberdeen, Washington

TRANSVERSE - NORTH BASIN TRESTLE PILE HORIZONTAL DISPLACEMENT

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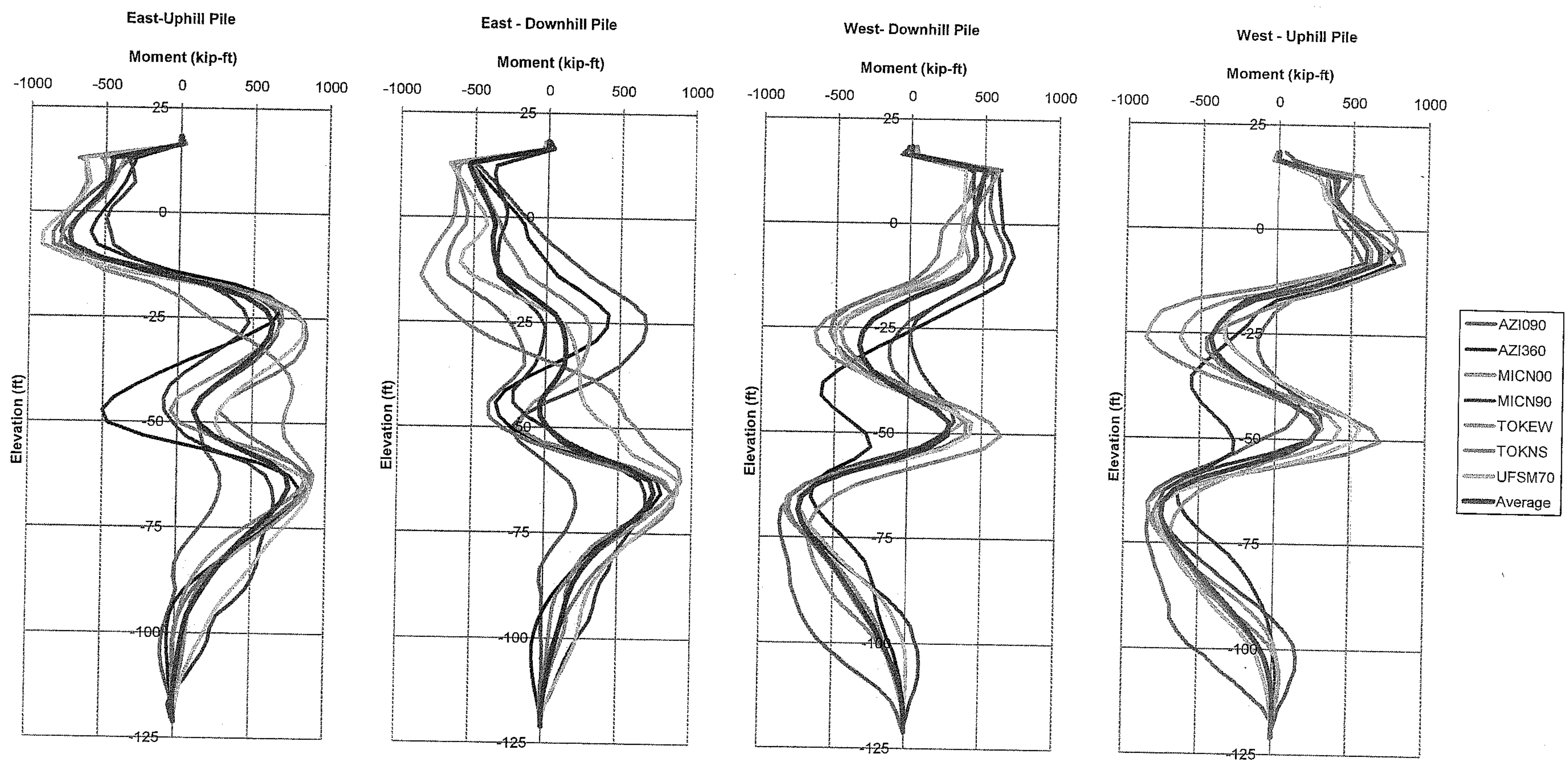
FIG. 20



NOTES

1. Pile response includes plastic hinging behavior. See text for plastic yield moments.
2. Results represent the effects of soil-structure interaction.
3. Results are taken at the end of ground motion shaking.

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Aberdeen, Washington

TRANSVERSE - NORTH BASIN TRESTLE PILE MOMENTS

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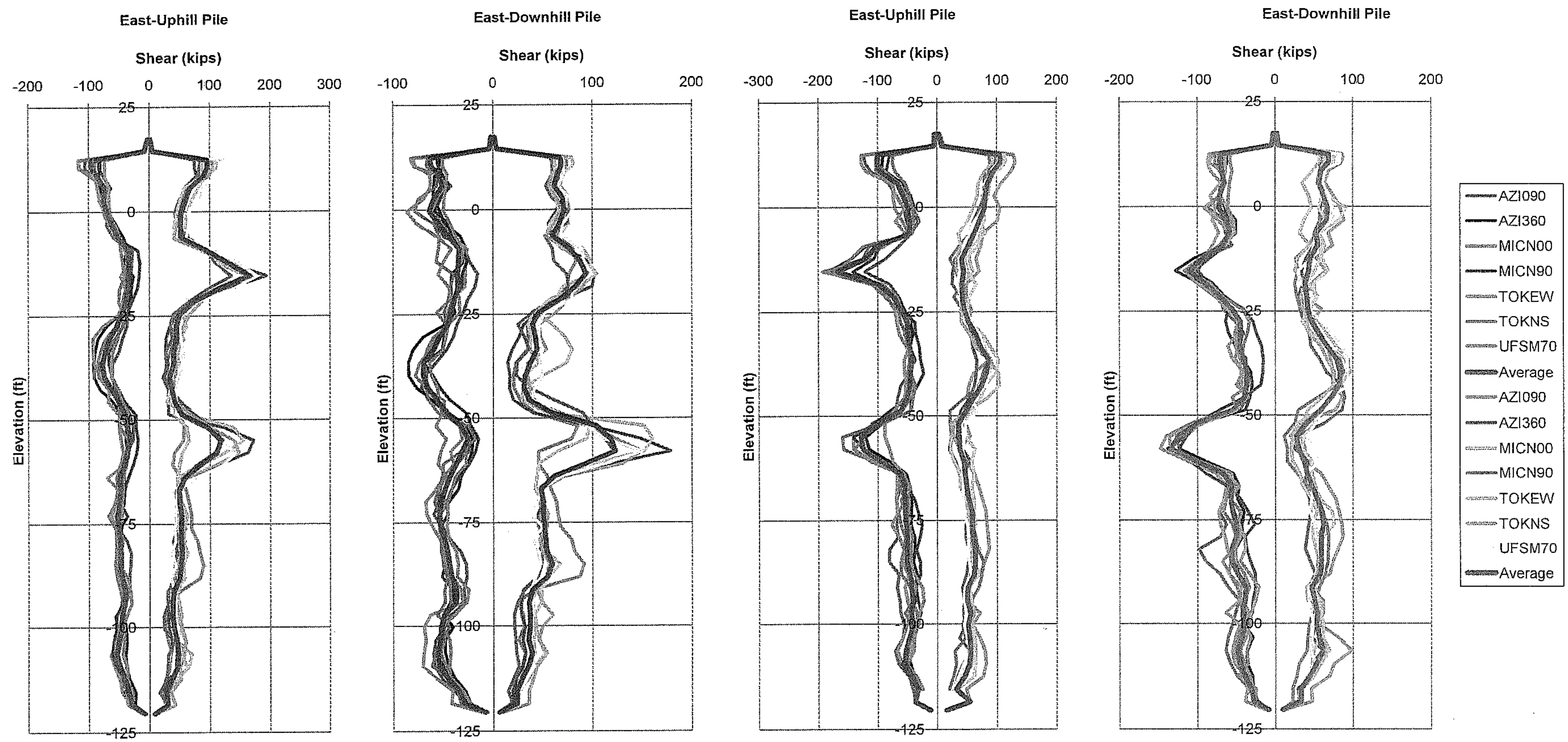
FIG. 21

Figures (North).xlsm Printed: 1/12/2011 4:49 PM



NOTES

1. Pile response includes plastic hinging behavior. See text for plastic yield moments.
2. Results represent the effects of soil-structure interaction.
3. Results represent the minimum and maximum shears recorded during shaking

SR 520 Pontoon Casting Facility
Aberdeen, Washington

TRANSVERSE - NORTH BASIN TRESTLE PILE MIN/MAX SHEAR

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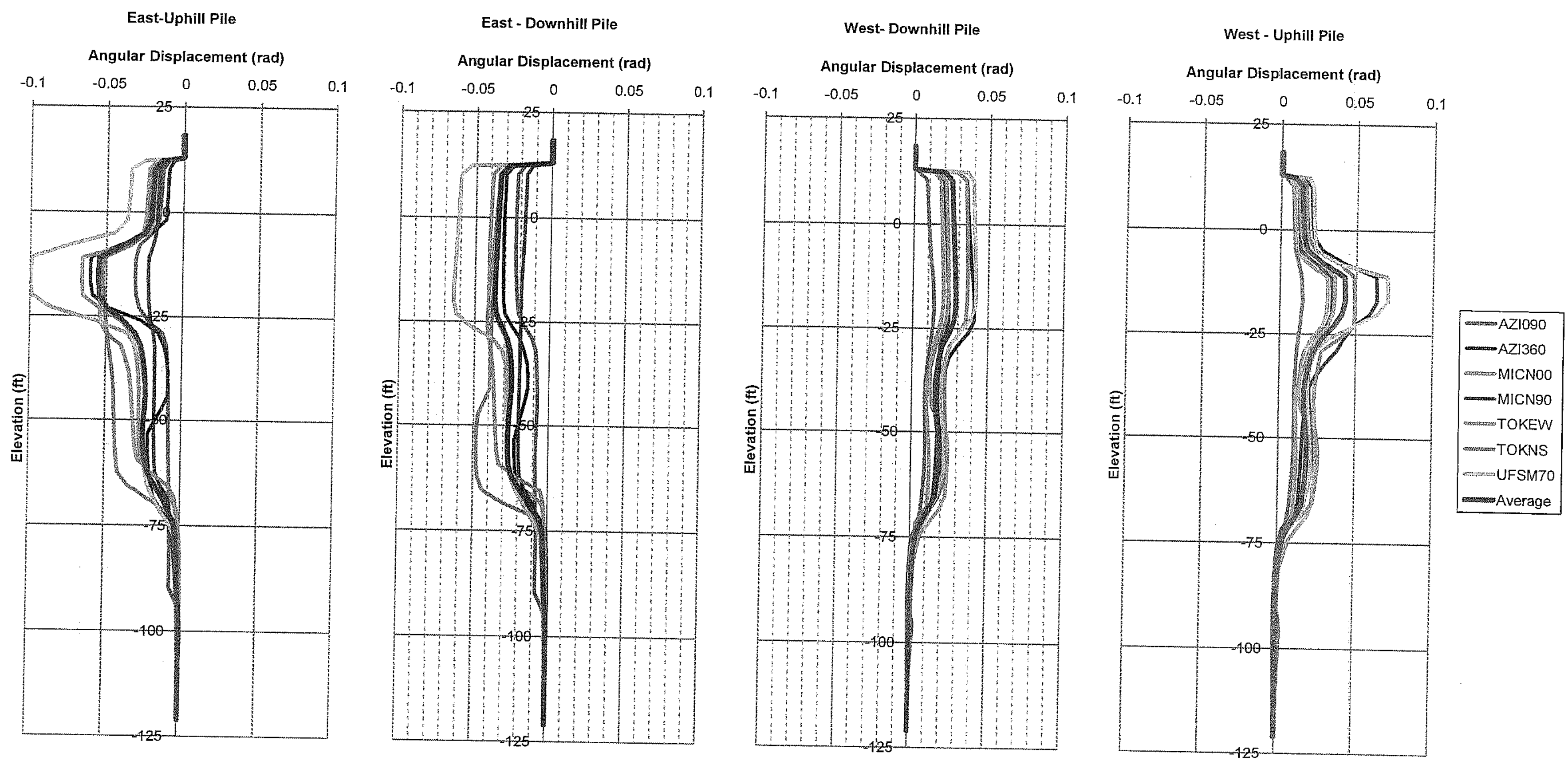
FIG. 22



NOTES

1. Pile response includes plastic hinging behavior. See text for plastic yield moments.
2. Results represent the effects of soil-structure interaction.
3. Results are taken at the end of ground motion shaking.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

TRANSVERSE - NORTH BASIN TRESTLE PILE ANGULAR DISPLACEMENTS

December 2010 21-1-21190-016

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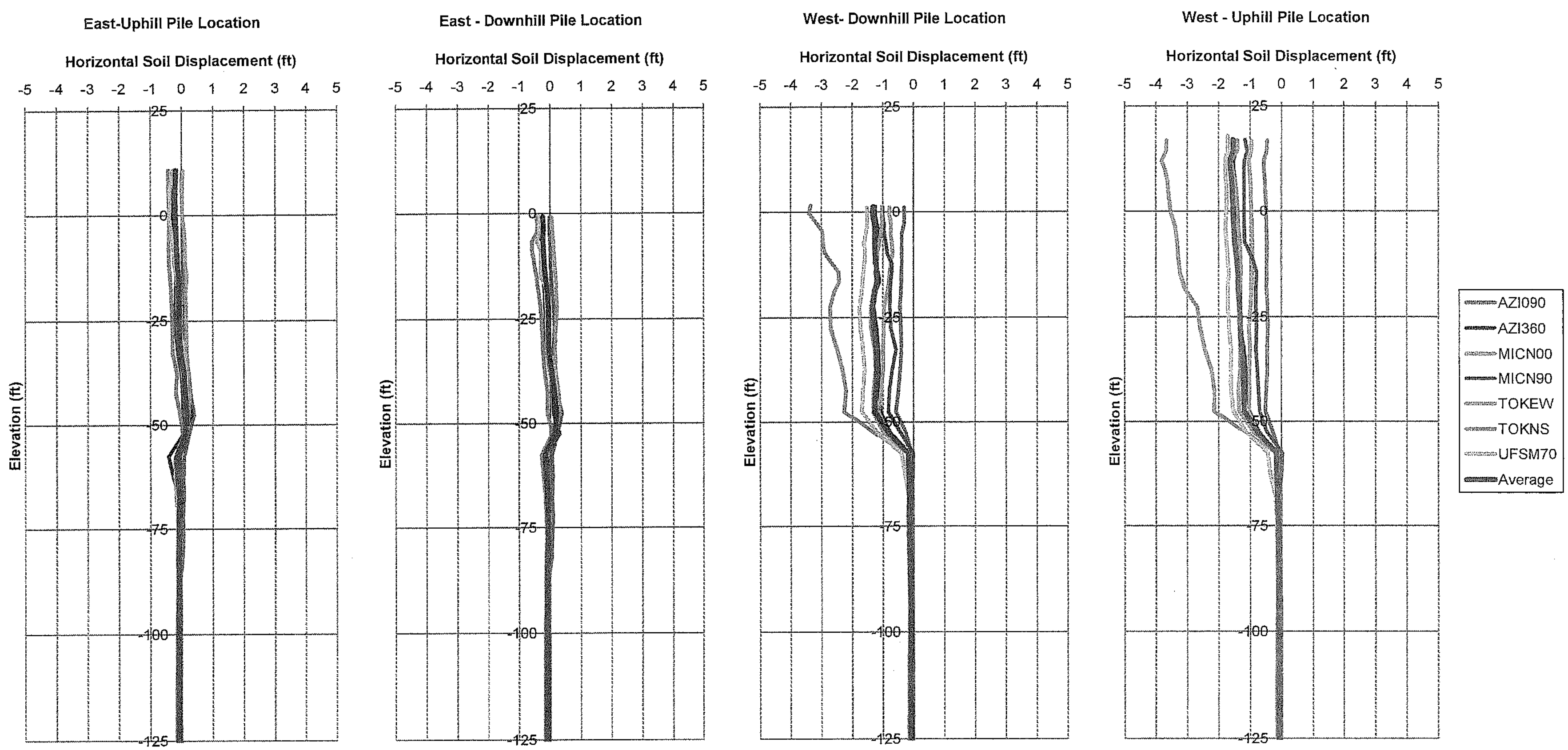
FIG. 23

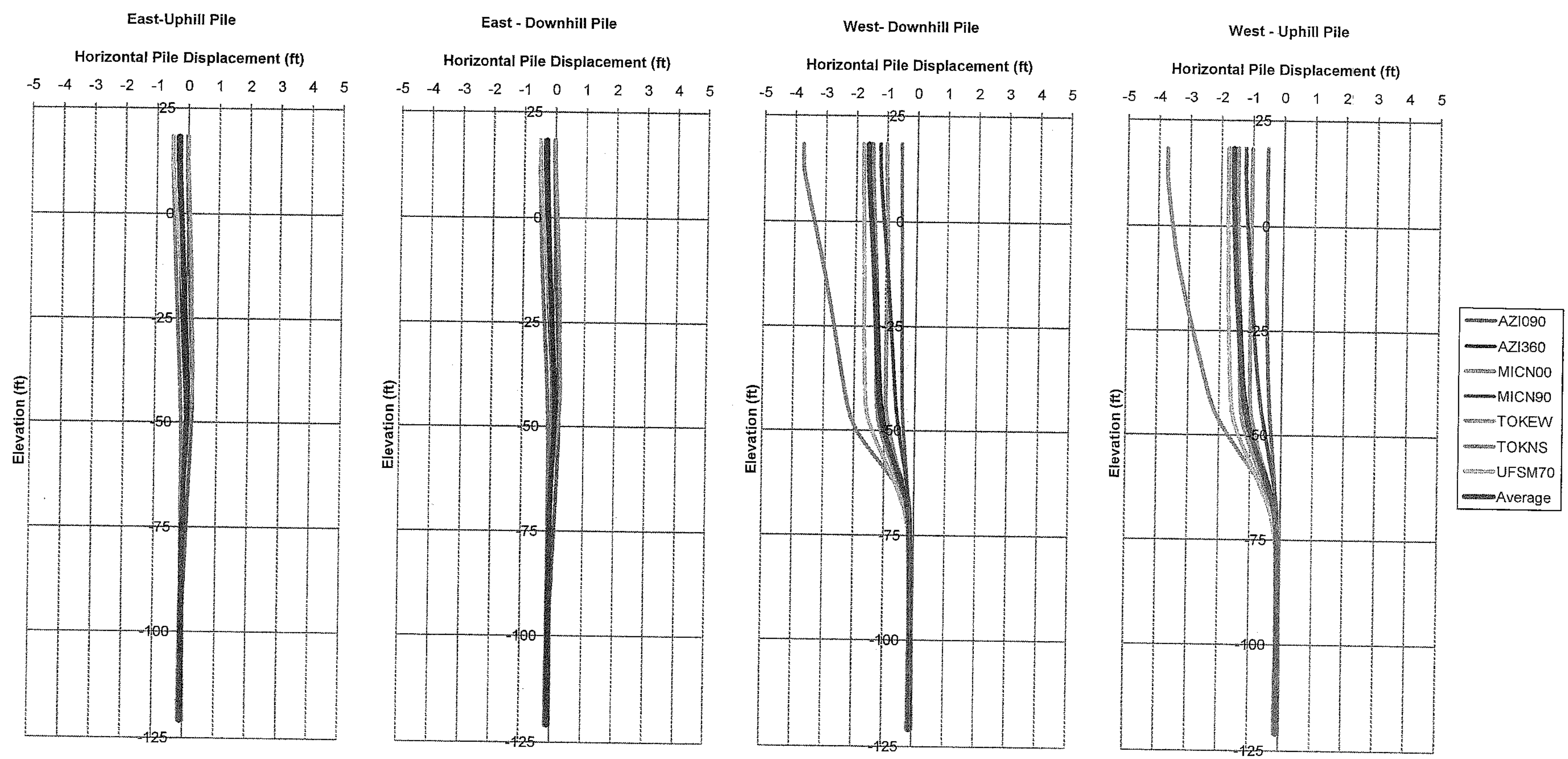
Figures (South).xlsm Printed: 12/30/2010 4:24 PM



NOTES

1. Ground displacements are near crane trestle pile loactions.
2. Results represent the effects of soil-structure interaction.
3. Results are taken at the end of ground motion shaking.
4. Positive displacments represent movements to the west and negative displacements represent movements to the east.

SR 520 Pontoon Casting Facility Aberdeen, Washington	
TRANSVERSE - SOUTH BASIN HORIZ. SOIL DISPLACEMENT	
December 2010	21-1-21190-016
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. 24



NOTES

1. Pile response includes plastic hinging behavior. See text for plastic yield moments.
2. Results represent the effects of soil-structure interaction.
3. Results are taken at the end of ground motion shaking.
4. Positive displacments represent movements to the west and negative displacements represent movements to the east.

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TRANSVERSE - SOUTH BASIN TRESTLE PILE HORIZONTAL DISPLACEMENT

December 2010 21-1-21190-016

SHANNON & WILSON, INC.
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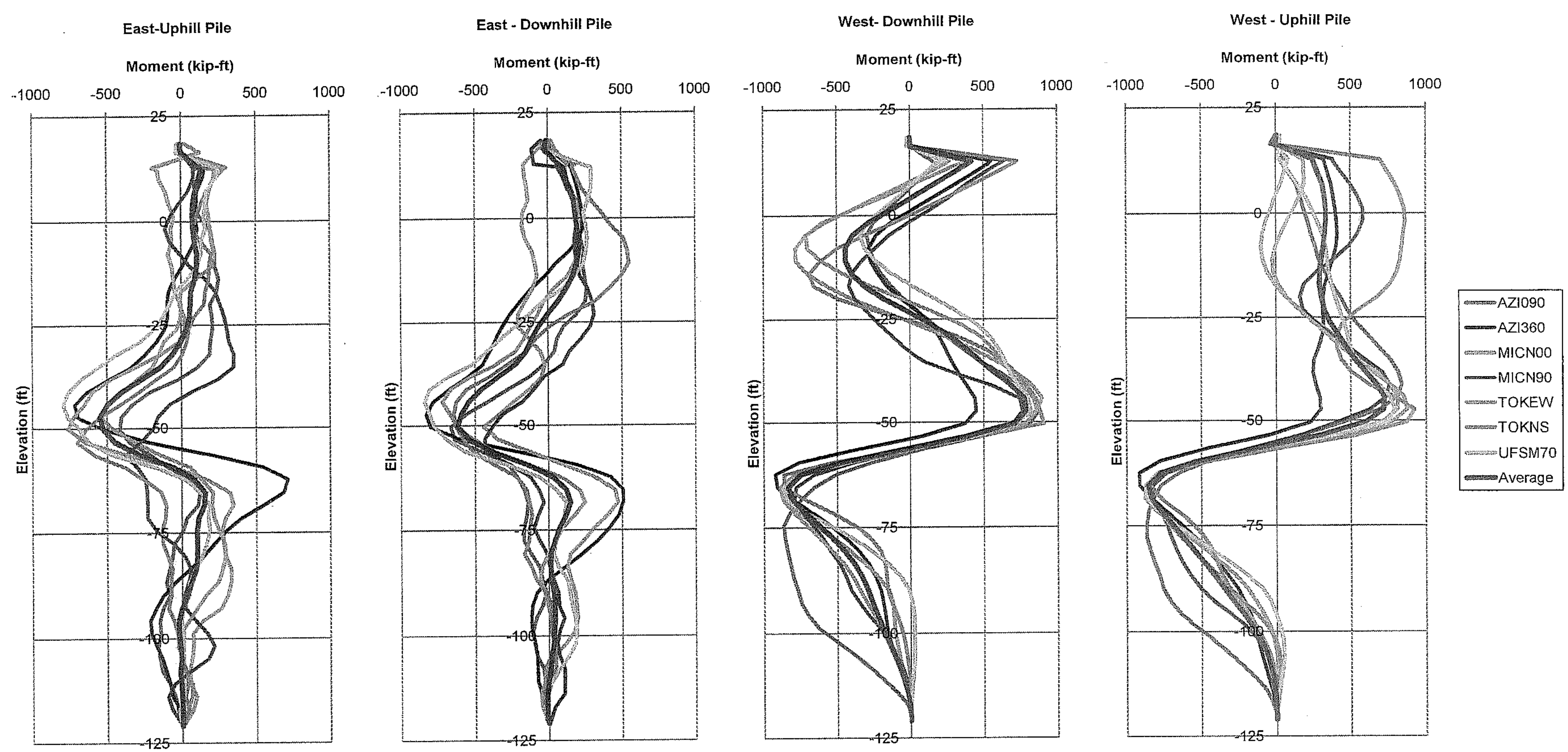
FIG. 25

Figures (South).xlsm Printed: 12/30/2010 4:24 PM



NOTES

1. Pile response includes plastic hinging behavior. See text for plastic yield moments.
2. Results represent the effects of soil-structure interaction.
3. Results are taken at the end of ground motion shaking.

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Aberdeen, Washington

TRANSVERSE - SOUTH BASIN
TRESTLE PILE MOMENTS

December 2010 21-1-21190-016

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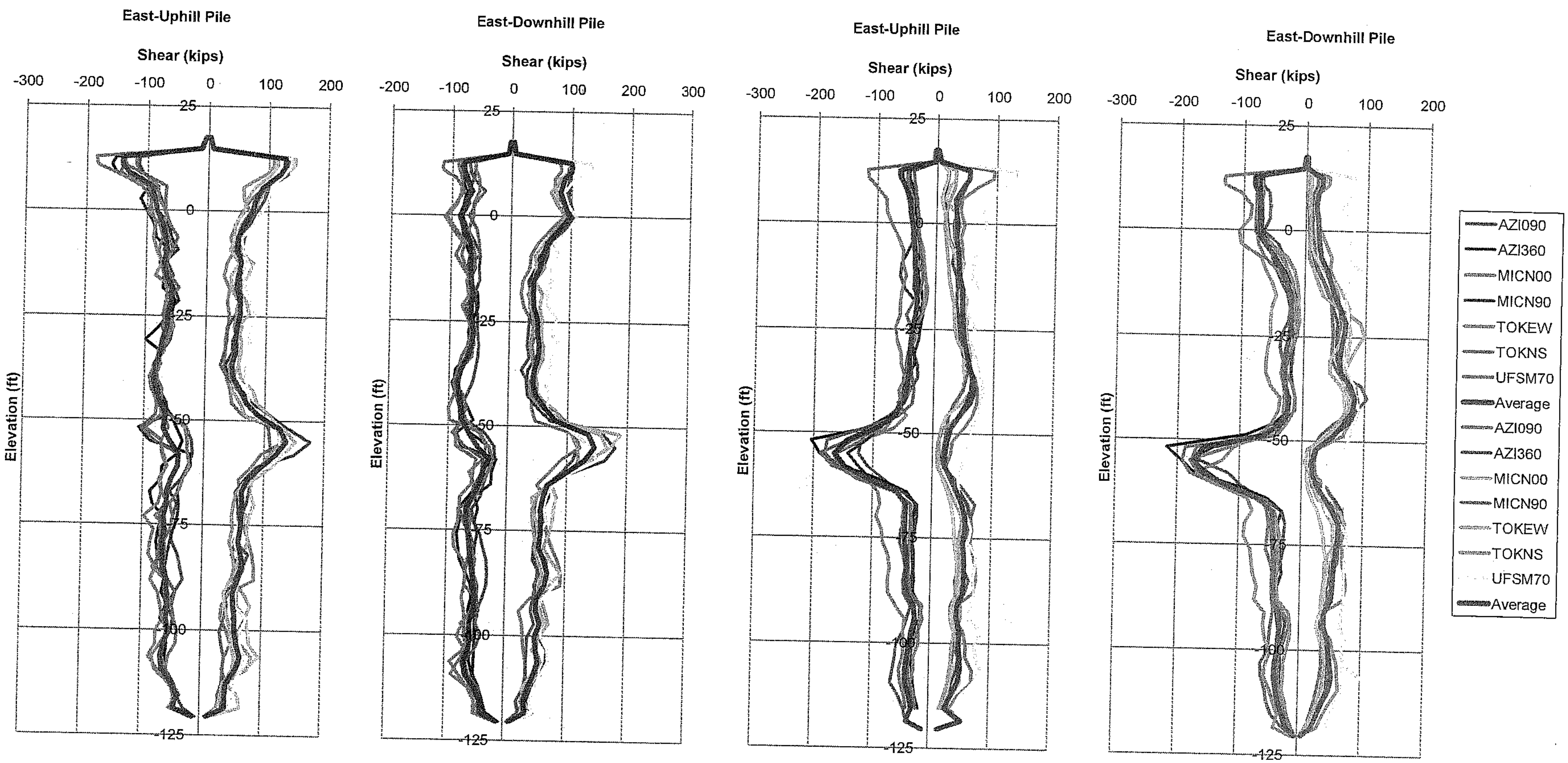
FIG. 26

Figures (South).xlsm Printed: 1/12/2011 3:39 PM



NOTES

1. Pile response includes plastic hinging behavior. See text for plastic yield moments.
2. Results represent the effects of soil-structure interaction.
3. Results represent the minimum and maximum shears recorded during shaking

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Aberdeen, Washington

TRANSVERSE - SOUTH BASIN TRESTLE PILE MIN/MAX SHEAR

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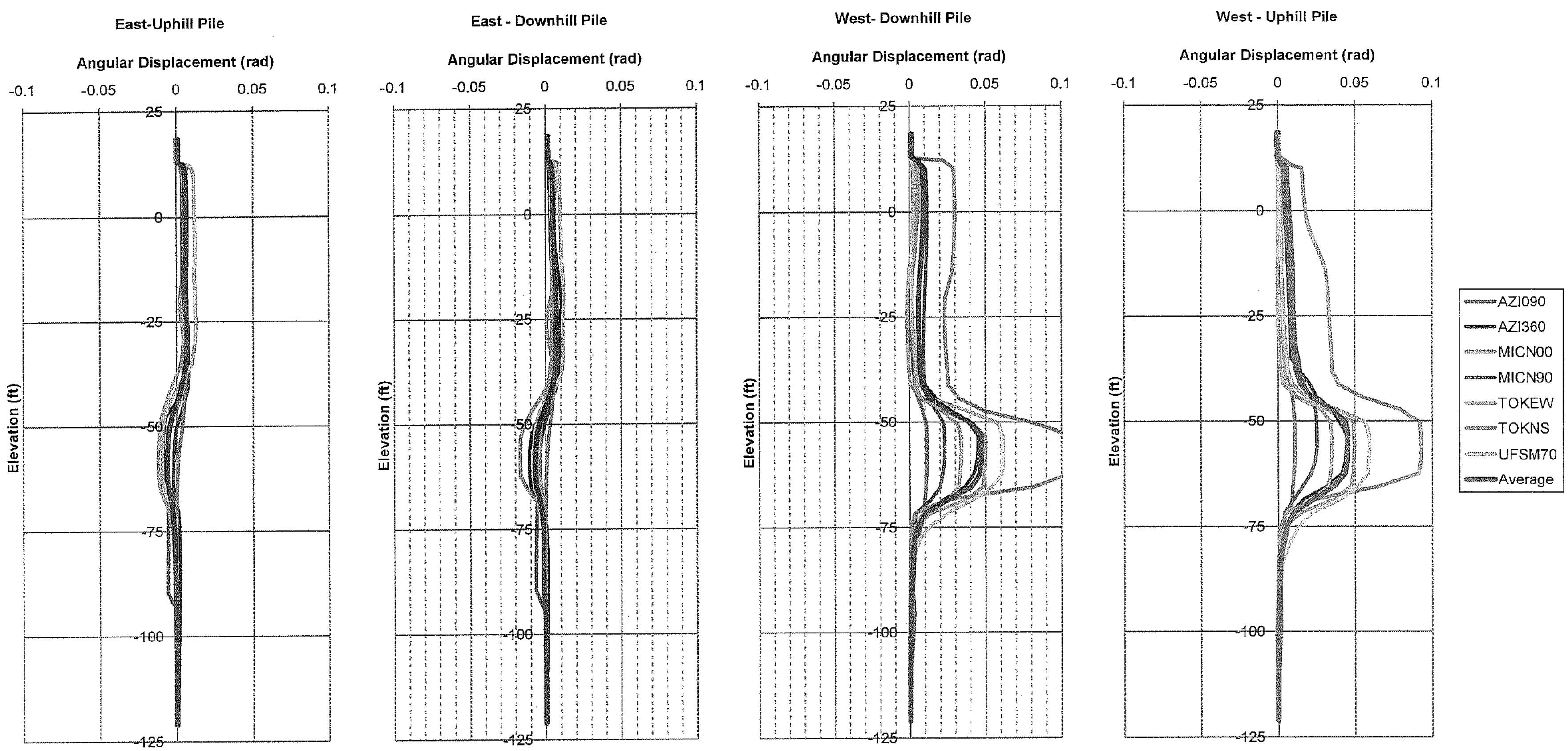
FIG. 27

Figures (South).xlsm Printed: 12/30/2010 4:24 PM



NOTES

1. Pile response includes plastic hinging behavior. See text for plastic yield moments.
2. Results represent the effects of soil-structure interaction.
3. Results are taken at the end of ground motion shaking.

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Aberdeen, Washington

TRANSVERSE - SOUTH BASIN TRESTLE PILE ANGULAR DISPLACEMENTS

December 2010 21-1-21190-016

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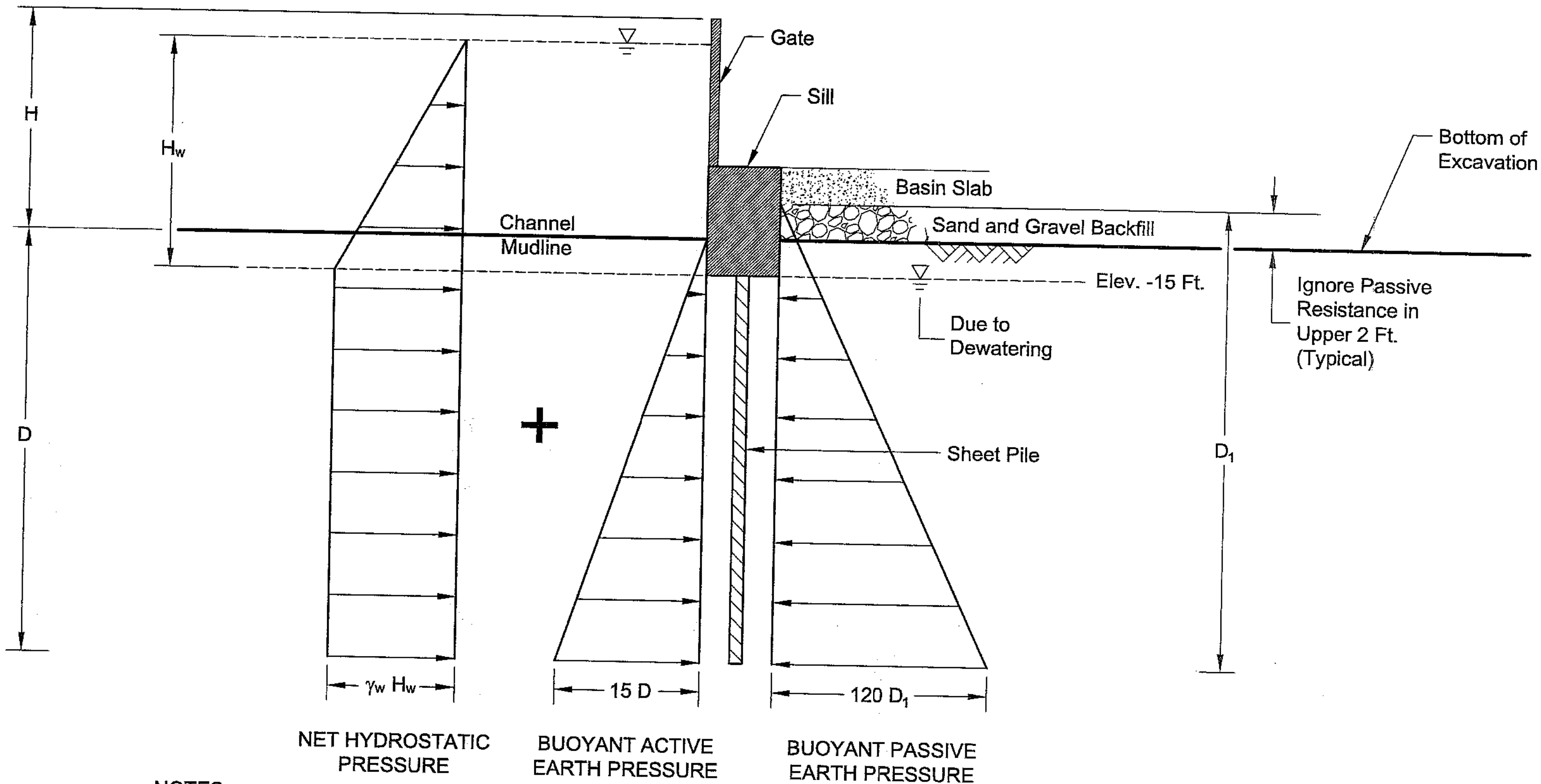
FIG. 28



Not to Scale

NOTES

1. All earth pressures are in units of pounds per square foot.
2. Use γ_w = 62.4 pcf. Refer to Figure H-14 for permanent dewatered groundwater level.
3. Passive earth pressures are ultimate values. Per AASHTO Table 11.5.6-1 (2008, Interim), the resistance factor for the strength limit case is 0.75.
4. The recommended pressure diagrams are based on a continuous wall system.
5. Structural features are schematic.
6. Additional lateral resistance would be provided by the piles supporting the sill.

LEGEND

H = Height Above Mudline (Ft.)

H_w = Differential Water Height (Ft.)

D = Embedment Below Mudline (Ft.)

D_1 = Embedment Below Basin Slab (Ft.)

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LATERAL EARTH PRESSURES FOR GATE CUTOFF WALL

December 2010 21-1-21190-015

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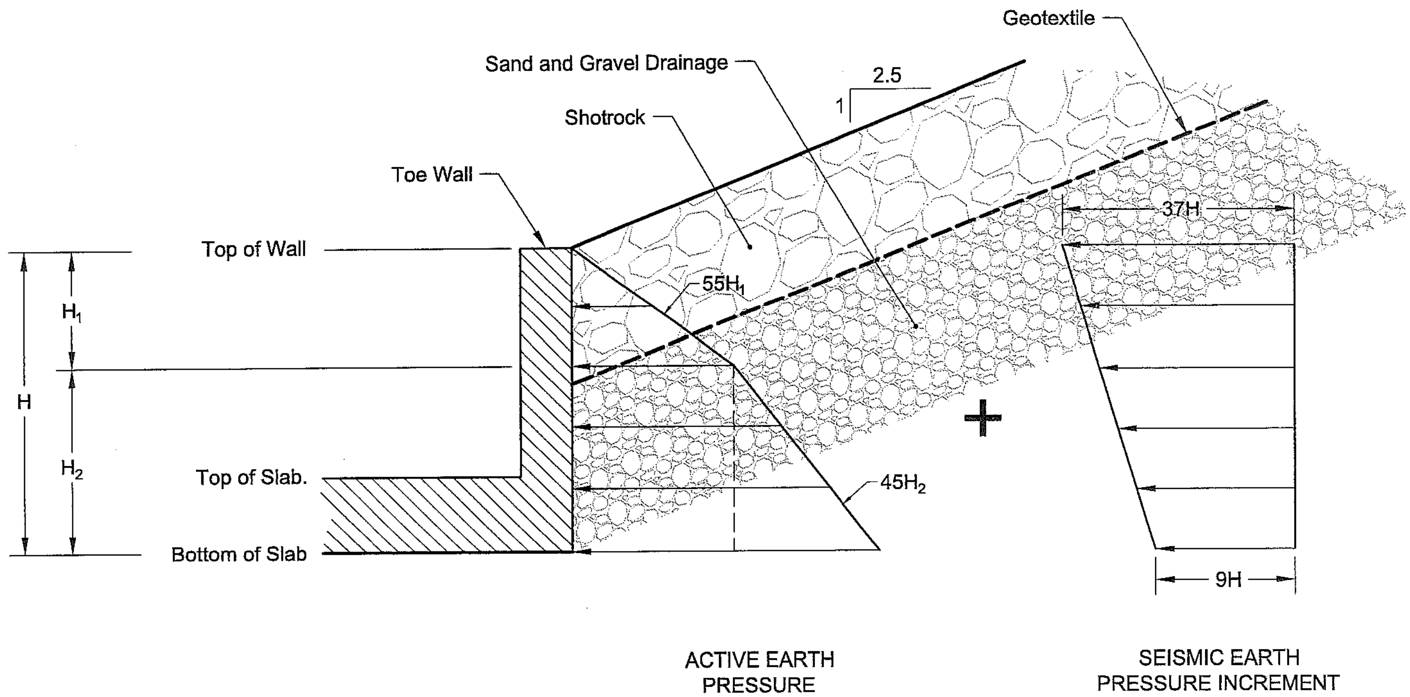
FIG. 29

FIG. 29

Filename: J:\211\21190-015\21-1-21190-015 Lateral Earth Pressure Diagrams.dwg Layout: Basin Toe Wall Date: 02-11-2011 Login: bac



Not to Scale

NOTES

1. All earth pressures are in units of pounds per square foot.
2. The recommended pressure diagrams are based on a continuous wall system.
3. Free drainage is assumed behind the wall.

LEGEND

H, H_1, H_2 = Height (Ft.)

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Aberdeen, Washington

LATERAL EARTH PRESSURES FOR BASIN TOE WALL

December 2010 21-1-21190-015

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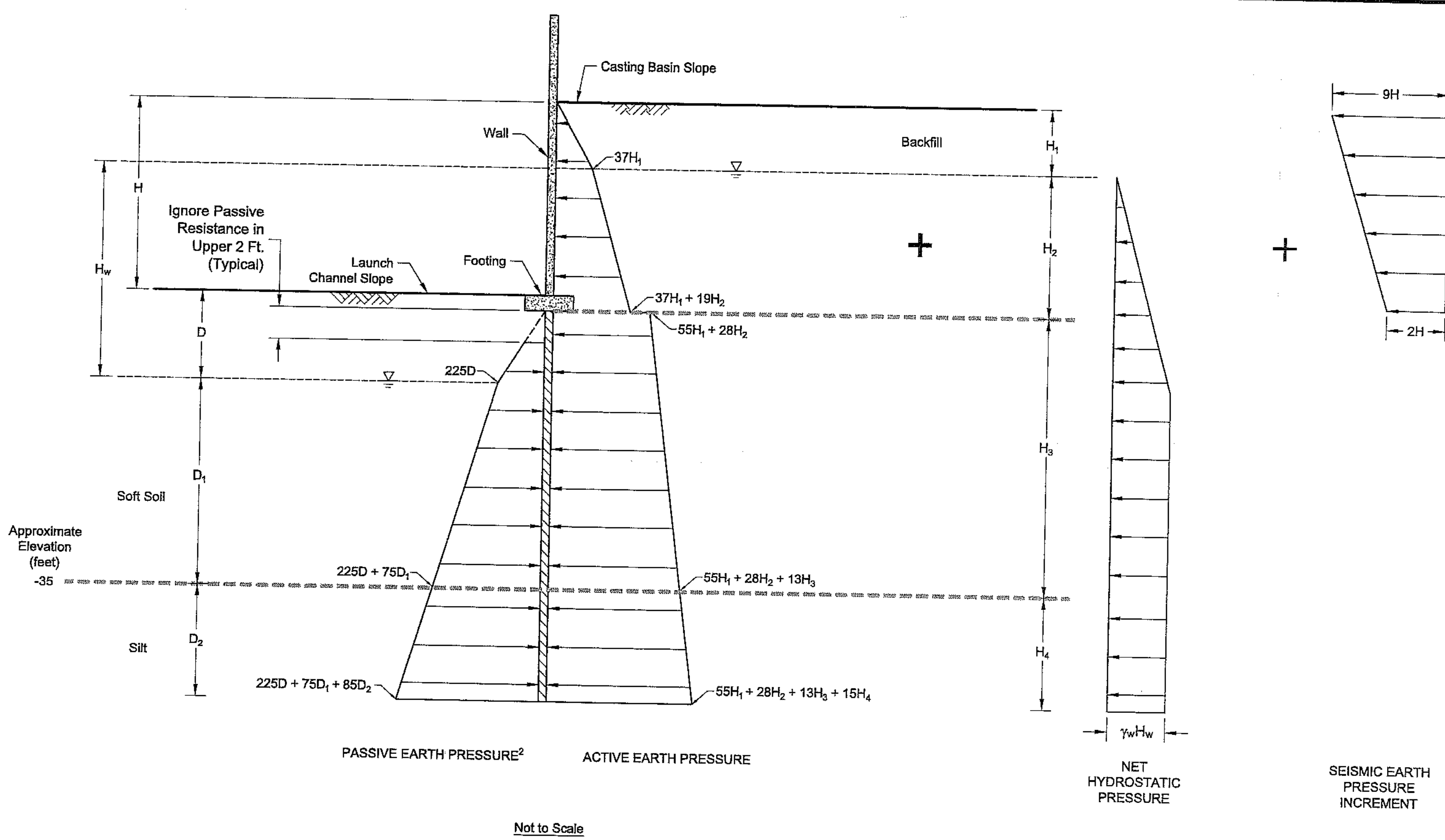
FIG. 30

FIG. 30



NOTES

1. All earth pressures are in units of pounds per square foot.
2. Passive pressures are ultimate values. Per AASHTO Table 11.5.6-1 (2008, Interim), the resistance factor for the strength limit case is 0.75.
3. The recommended pressure diagrams are based on a continuous wall system.
4. Additional lateral resistance would be provided by the piles supporting the bulkhead.
5. The water levels on the basin and channel sides will vary above and below the ground surface depending upon the position of the section and load case. Use dry pressure values above the water surface and saturated pressure values below the water surface.
6. Use γ_w = 62.4 pcf. Refer to Figure H-14 for permanent dewatered groundwater level.

LEGEND

H = Excavation Height (Ft.)

H_1 = Height of Soil Above Water (Ft.)

H_2, H_3, H_4 = Height of Soil Below Water (Ft.)

H_w = Water Height Differential (Ft.)

D = Embedment Depths Above Water (Ft.)

D_1, D_2 = Embedment Depths Below Water (Ft.)

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Aberdeen, Washington

LATERAL EARTH PRESSURES FOR GATE BULKHEAD AND SHEET PILE WALL (STATIC/SEISMIC)

December 2010 | 21-1-21190-015

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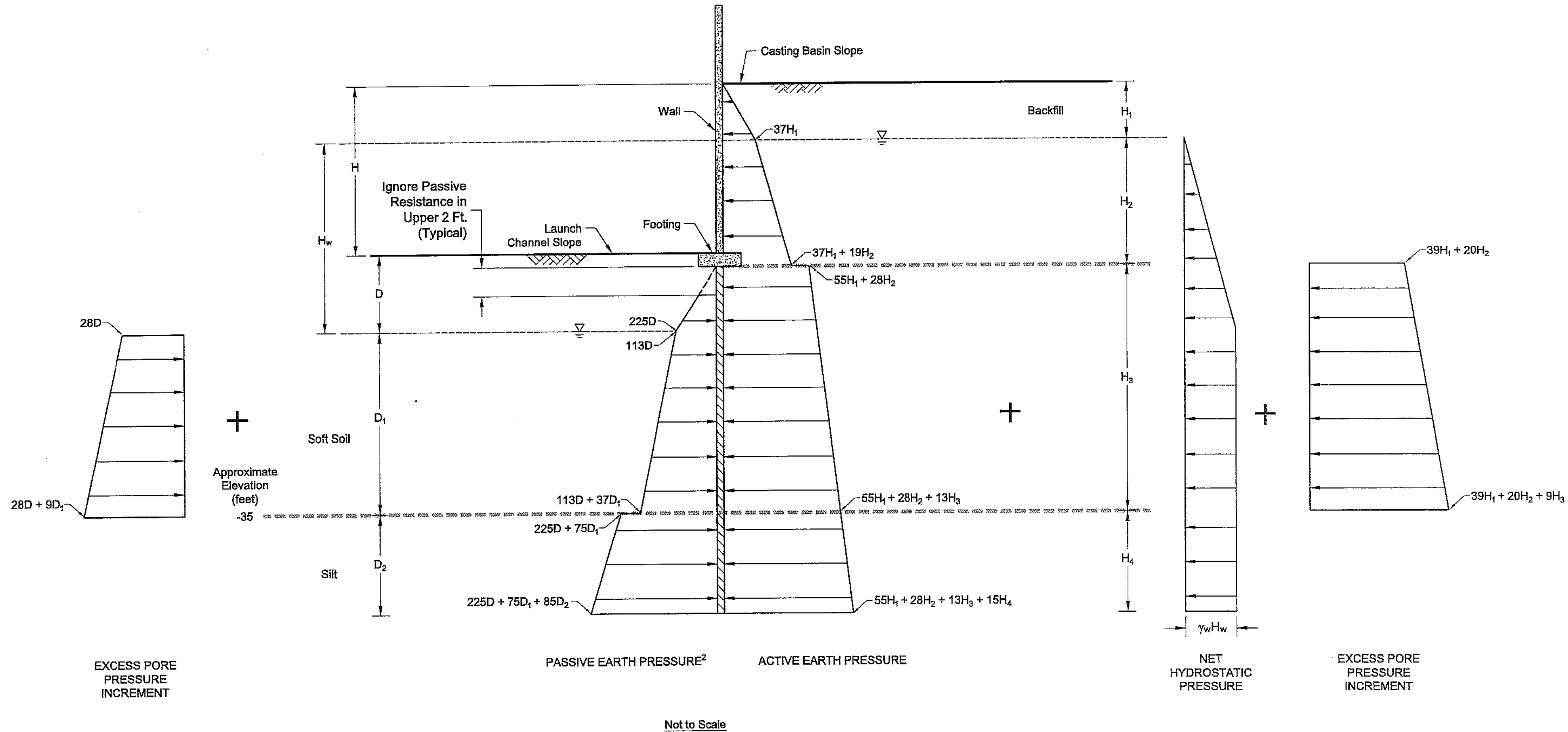
FIG. 31A

Filename: J:\211\21190-015\21-1-21190-015 Lateral Earth Pressure Diagrams.dwg Layout: Gate Bulkhead and Sheet Pile Wall Static-Seismic Date: 02-11-2011 Login: bac



NOTES

1. All earth pressures are in units of pounds per square foot.
2. Passive pressures are ultimate values. Per AASHTO Table 11.5.6-1 (2008, Interim), the resistance factor for the strength limit case is 0.75.
3. The recommended pressure diagrams are based on a continuous wall system.
4. Additional lateral resistance would be provided by the piles supporting the bulkhead.
5. The water levels on the basin and channel sides will vary above and below the ground surface depending upon the position of the section and load case. Use dry pressure values above the water surface and saturated pressure values below the water surface.
6. Use γ_w = 62.4 pcf. Refer to Figure H-14 for permanent dewatered groundwater level.

LEGEND

H = Excavation Height (Ft.)

H_1 = Height of Soil Above Water (Ft.)

H_2, H_3, H_4 = Height of Soil Below Water (Ft.)

H_w = Water Height Differential (Ft.)

D = Embedment Depths Above Water (Ft.)

D_1, D_2 = Embedment Depths Below Water (Ft.)

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LATERAL EARTH PRESSURES FOR GATE BULKHEAD AND SHEET PILE WALL (POST-SEISMIC)

December 2010 21-1-21190-015

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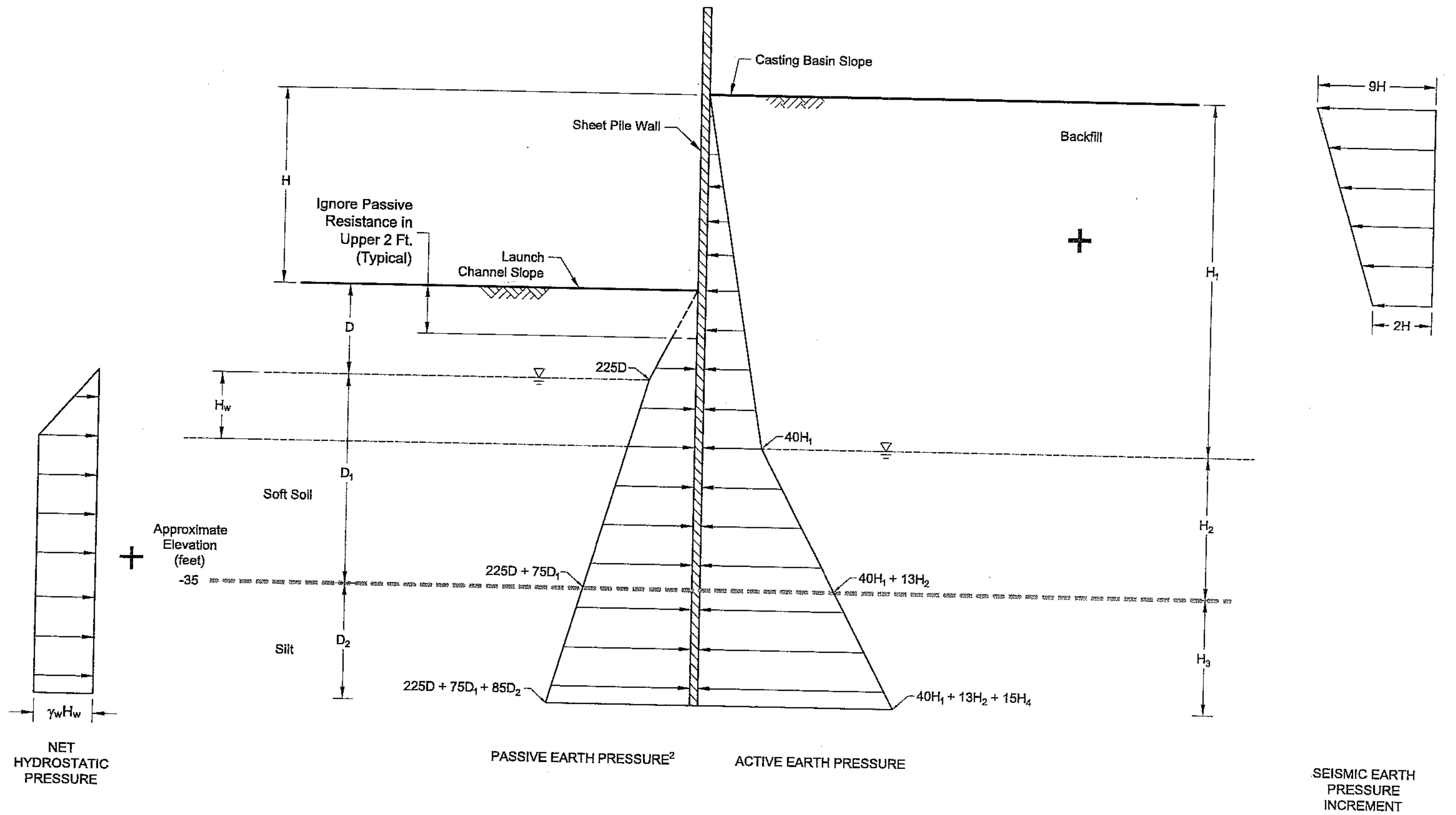
FIG. 31B



Not to Scale

NOTES

1. All earth pressures are in units of pounds per square foot.
2. Passive pressures are ultimate values. Per AASHTO Table 11.5.6-1 (2008, Interim), the resistance factor for the strength limit case is 0.75.
3. The recommended pressure diagrams are based on a continuous wall system.
4. The water levels on the basin and channel sides will vary above and below the ground surface depending upon the position of the section and load case. Use dry pressure values above the water surface and saturated pressure values below the water surface.
5. Use γ_w = 62.4 pcf. Refer to Figure H-14 for permanent dewatered groundwater level.

LEGEND

H = Excavation Height (Ft.)

H_1 = Height of Soil Above Water (Ft.)

H_2, H_3 = Height of Soil Below Water (Ft.)

H_w = Water Height Differential (Ft.)

D = Embedment Depths Above Water (Ft.)

D_1, D_2 = Embedment Depths Below Water (Ft.)

SR 520 Pontoon Casting Facility
Aberdeen, Washington

LATERAL EARTH PRESSURES FOR GATE BULKHEAD AND SHEET PILE WALL (STATIC/SEISMIC)

December 2010 21-1-21190-015

SHANNON & WILSON, INC.
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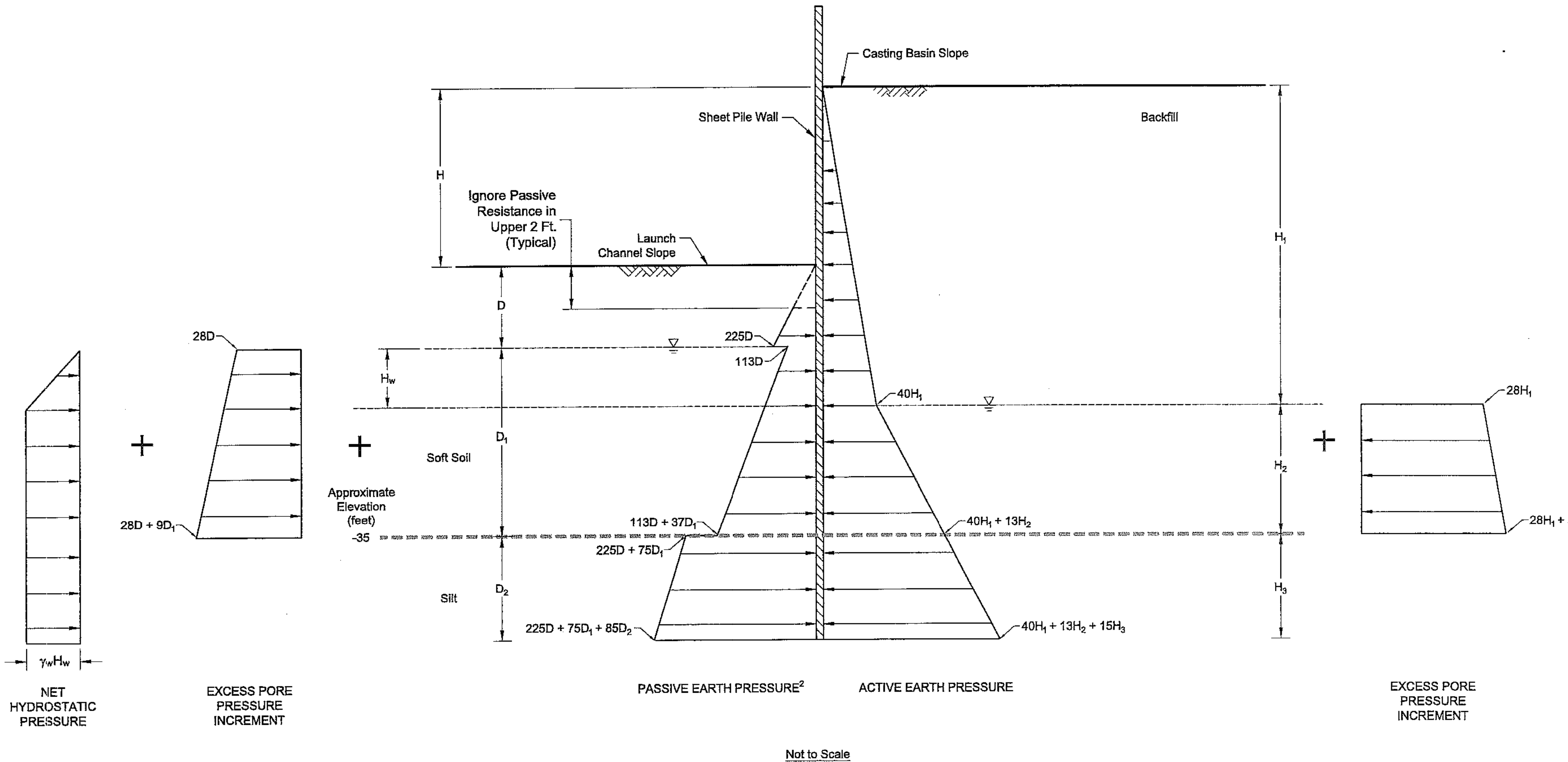
FIG. 31C



NOTES

1. All earth pressures are in units of pounds per square foot.
2. Passive pressures are ultimate values. Per AASHTO Table 11.5.6-1 (2008, Interim), the resistance factor for the strength limit case is 0.75.
3. The recommended pressure diagrams are based on a continuous wall system.
4. The water levels on the basin and channel sides will vary above and below the ground surface depending upon the position of the section and load case. Use dry pressure values above the water surface and saturated pressure values below the water surface.
5. Use γ_w = 62.4 pcf. Refer to Figure H-14 for permanent dewatered groundwater level.

LEGEND

H = Excavation Height (Ft.)

H_1 = Height of Soil Above Water (Ft.)

H_2, H_3 = Height of Soil Below Water (Ft.)

H_w = Water Height Differential (Ft.)

D = Embedment Depths Above Water (Ft.)

D_1, D_2 = Embedment Depths Below Water (Ft.)

SR 520 Pontoon Casting Facility
Aberdeen, Washington

LATERAL EARTH PRESSURES FOR GATE BULKHEAD AND SHEET PILE WALL (POST-SEISMIC)

December 2010 21-1-21190-015

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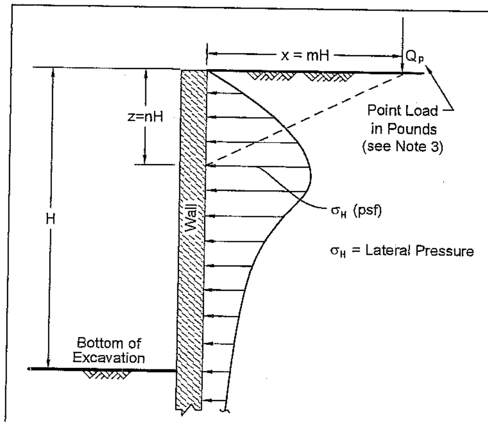
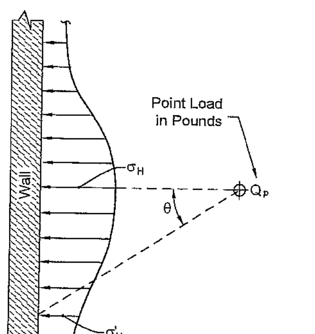
FIG. 31D



ELEVATION VIEW

For $m \leq 0.4$: $\sigma_H = 0.28 \dfrac{Q_P}{H^2} \dfrac{n^2}{(0.16 + n^2)^3}$ (psf) (see Note 3)

For $m > 0.4$: $\sigma_H = 1.77 \dfrac{Q_P}{H^2} \dfrac{m^2\, n^2}{(m^2 + n^2)^3}$ (psf)

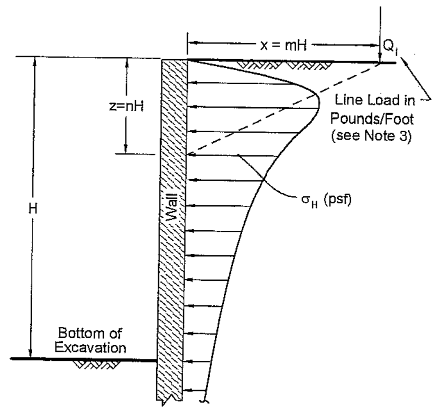


$\sigma'_H = \sigma_H \cos^2 (1.1\,\theta)$ (psf)

PLAN VIEW

A) LATERAL PRESSURE DUE TO POINT LOAD
i.e. SMALL ISOLATED FOOTING OR WHEEL LOAD

(NAVFAC DM 7.02, 1986)



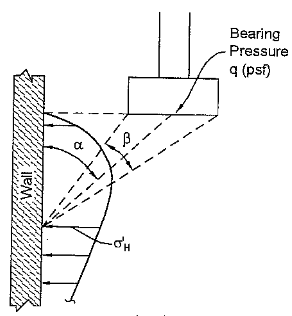
ELEVATION VIEW

For $m \leq 0.4$: $\sigma_H = 0.20 \dfrac{Q_l}{H} \dfrac{n}{(0.16 + n^2)^2}$ (psf) (see Note 3)

For $m > 0.4$: $\sigma_H = 1.28 \dfrac{Q_l}{H} \dfrac{m^2\, n}{(m^2 + n^2)^2}$ (psf)

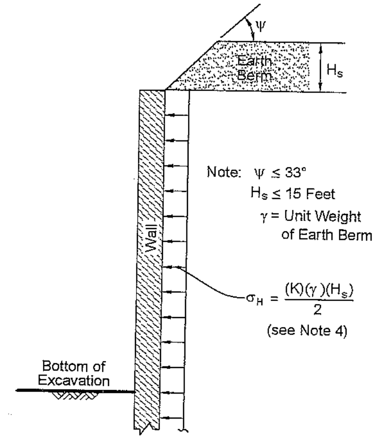
B) LATERAL PRESSURE DUE TO LINE LOAD
i.e. NARROW CONTINUOUS FOOTING
PARALLEL TO WALL

(NAVFAC DM 7.02, 1986)

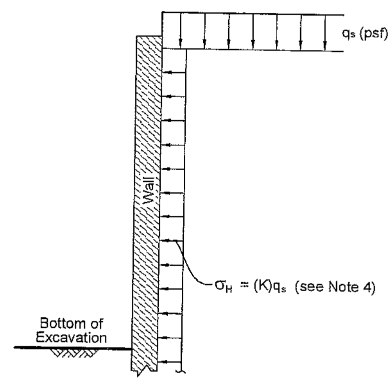


$\sigma_H = \dfrac{2q}{\pi} (\beta - \sin \beta \cos 2\alpha)$ (psf) — in radians

C) LATERAL PRESSURE DUE TO STRIP LOAD
(derived from Fang, *Foundation Engineering Handbook*, 1991)



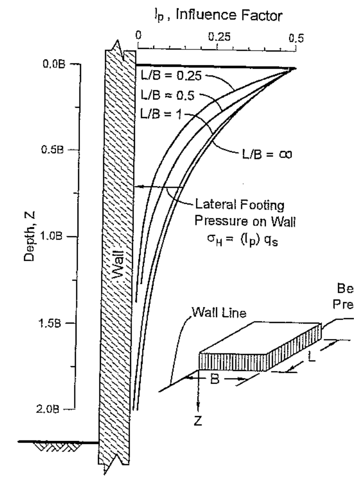
EARTH BERM



UNIFORM SURCHARGE

D) LATERAL PRESSURE DUE TO EARTH BERM
OR UNIFORM SURCHARGE

(derived from Poulos and Davis, *Elastic Solutions for Soil and Rock Mechanics*, 1974; and Terzaghi and Peck, *Soil Mechanics in Engineering Practice*, 1967)



E) LATERAL PRESSURE DUE
TO ADJACENT FOOTING

(derived from NAVFAC DM 7.02, 1986; and Sandhu, *Earth Pressure on Walls Due to Surcharge*, 1974)

The stress is estimated on the back of the wall at the edge of the footing. Assumes a Poisson's ratio of 0.5.

NOTES

1. Figures are not drawn to scale.
2. Applicable surcharge pressures should be added to appropriate permanent wall lateral earth and water pressure.
3. If point or line loads are close to the back of the wall such that $m \leq 0.4$, it may be more appropriate to model the actual load distribution (i.e., Detail E) or use more rigorous analysis methods.
4. Use: K = 0.28 for Backfill
K = 0.45 for Native Soil

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RECOMMENDED SURCHARGE LOADING FOR TEMPORARY AND PERMANENT WALLS

December 2010 21-1-21190-015

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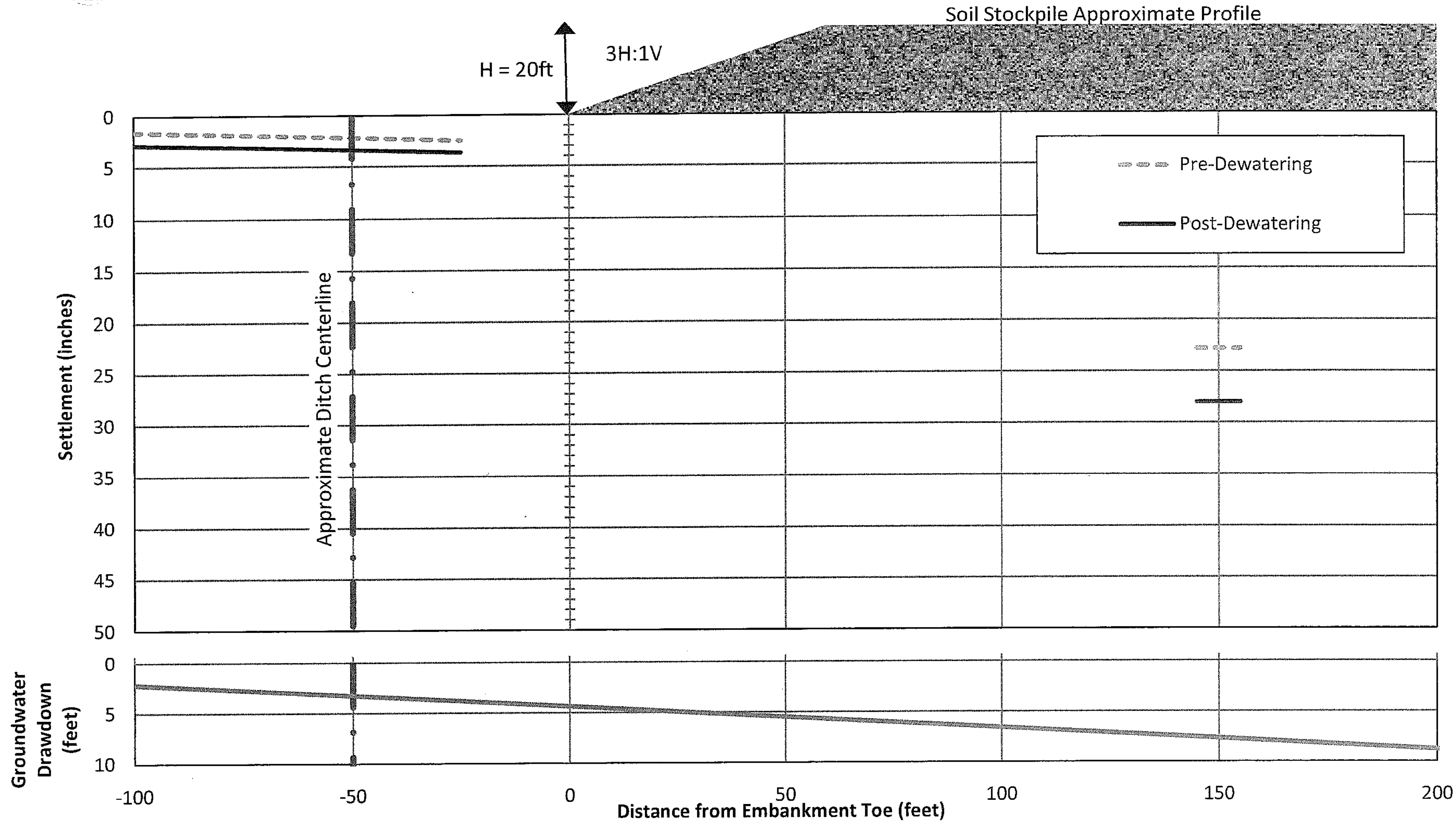
FIG. 32

File: J:\211\21190-015\21-1-21190-015 Surcharge Loading.dwg Date: 02-08-2011 Author: sac

Fig xx (idation Settlement 20ft.xlsm 8/24/2010



NOTES

1. See Appendix H for groundwater drawdown analyses.
2. We estimated the settlement using elastic stress distributions from Poulos and Davis (1973) and consolidation calculations.

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SETTLEMENT DUE TO SOIL STOCKPILE AND CASTING BASIN DEWATERING 20-FOOT HIGH SOIL STOCKPILE

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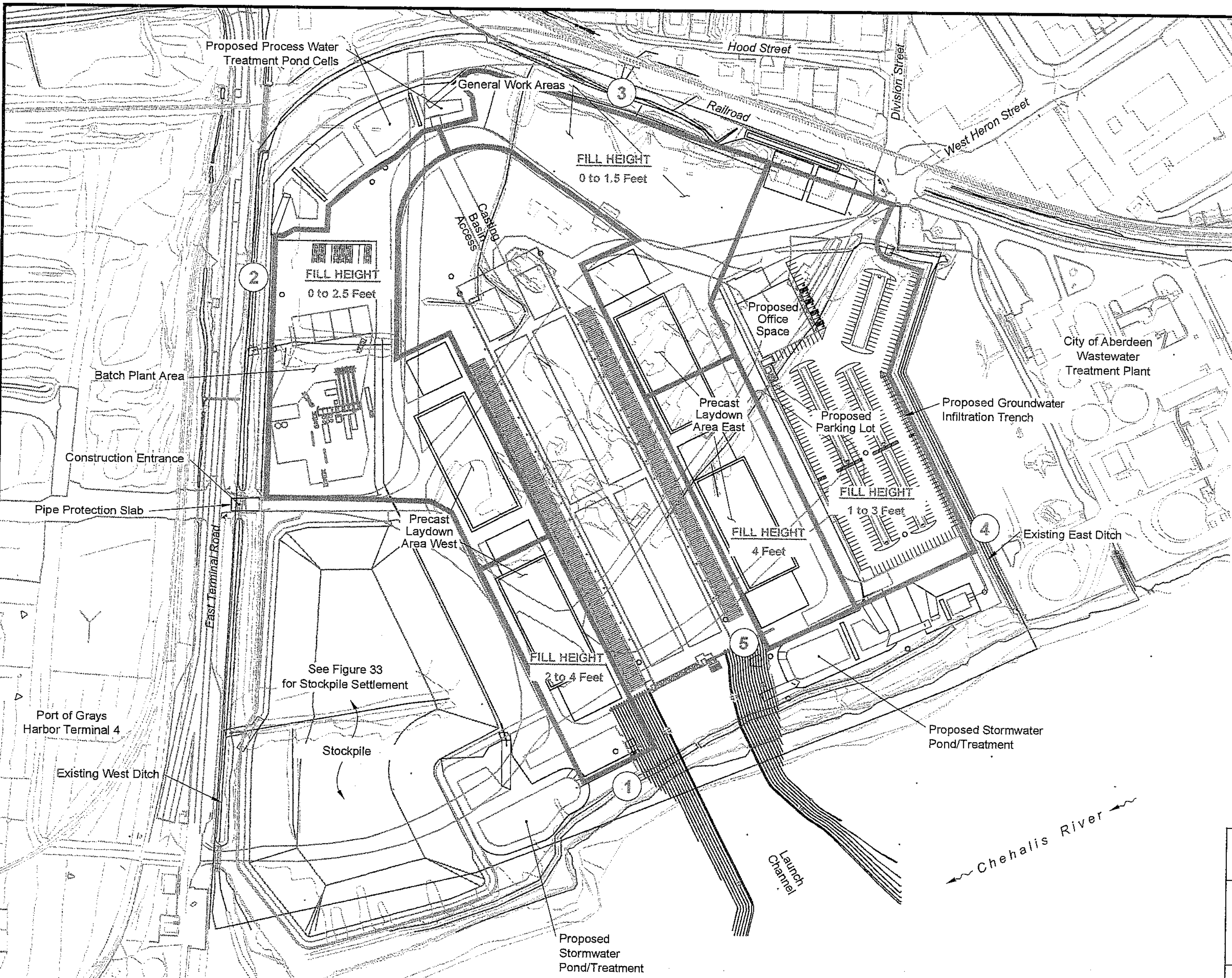
FIG. 33

FIG. 33



LEGEND

(1) Settlement Zone Designation

ZONE	ESTIMATED SETTLEMENT (See Note 1)
1	2 to 4 inches
2	1 to 3 inches
3	1 to 3 inches
4	1 to 4 inches
5	3 to 5 inches

NOTES

1. Settlement estimates include effects of fill placement and dewatering drawdown.
2. This figure is based on *XL2672_AL_BP_Base Map-Default.dwg* and, *Figure 3-10.dwg*, received 5-10, and *49854_GEO_s_001.dgn*, received 7-26-10.

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Aberdeen, Washington

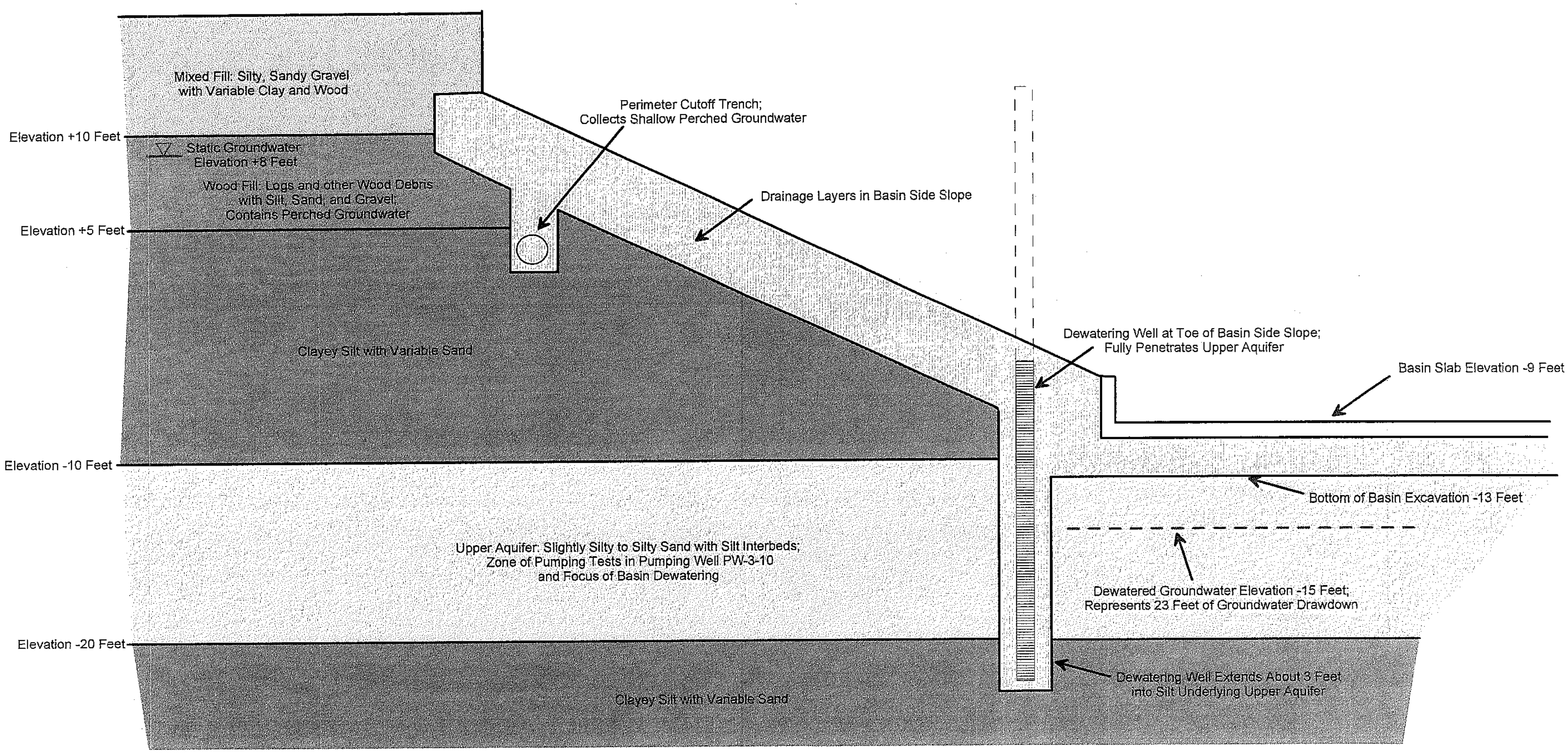
PROPOSED GRADING PLAN AND ESTIMATED SETTLEMENT

December 2010 21-1-21190-015

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

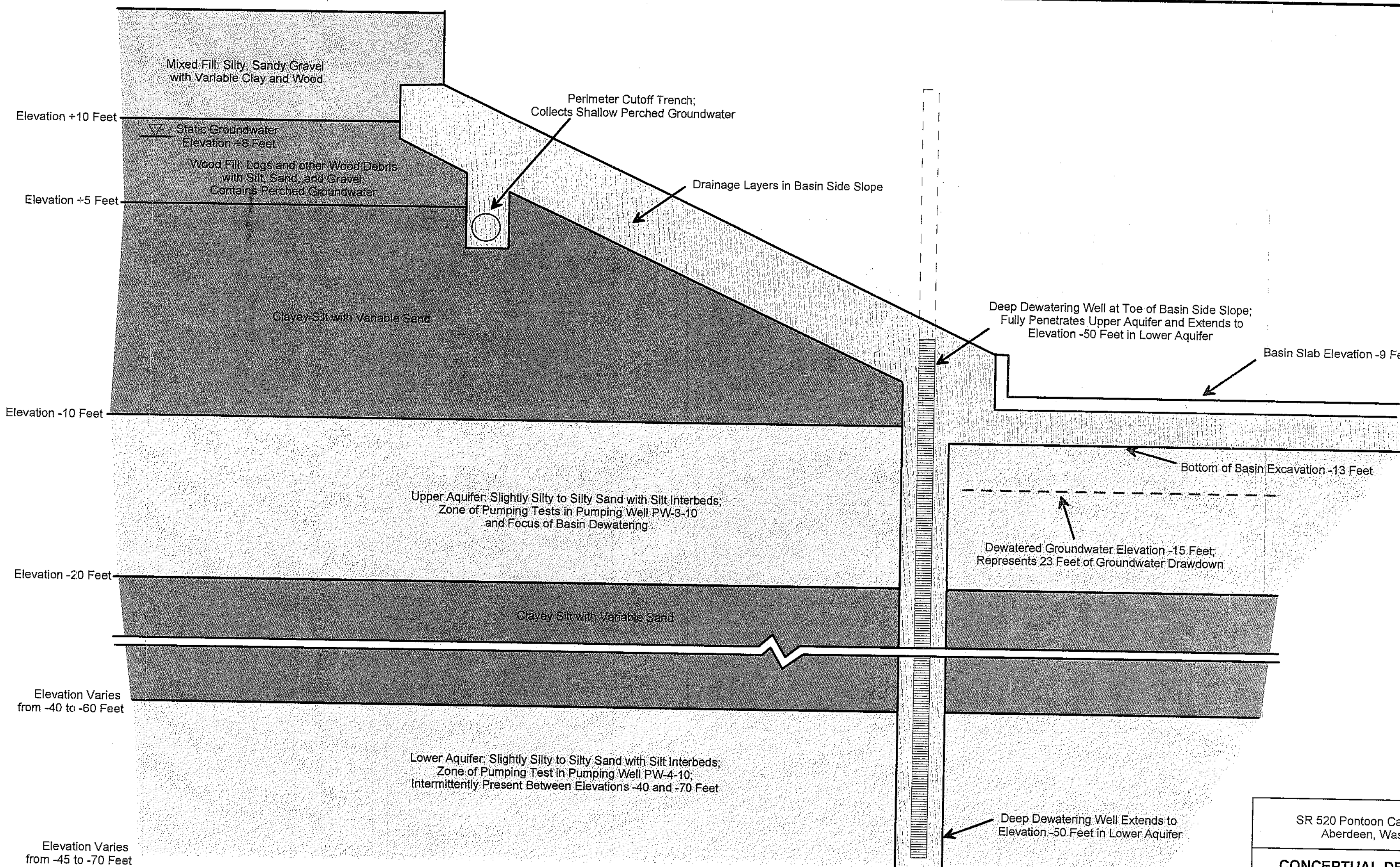
FIG. 34

Filename: J:\211\21190-015\21-1-21190-015 Grading Plan and Settlement.dwg Date: 02-11-2011 Login: KXW



Not to Scale
Dimensions and Elevations are Approximate

SR 520 Pontoon Casting Facility Aberdeen, Washington	
CONCEPTUAL DEWATERING CROSS SECTION UPPER AQUIFER	
December 2010	21-1-21190-015
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. 35**



Not to Scale
Dimensions and Elevations are Approximate

SR 520 Pontoon Casting Facility
Aberdeen, Washington

CONCEPTUAL DEWATERING CROSS SECTION LOWER AQUIFER

December 2010 21-1-21190-015

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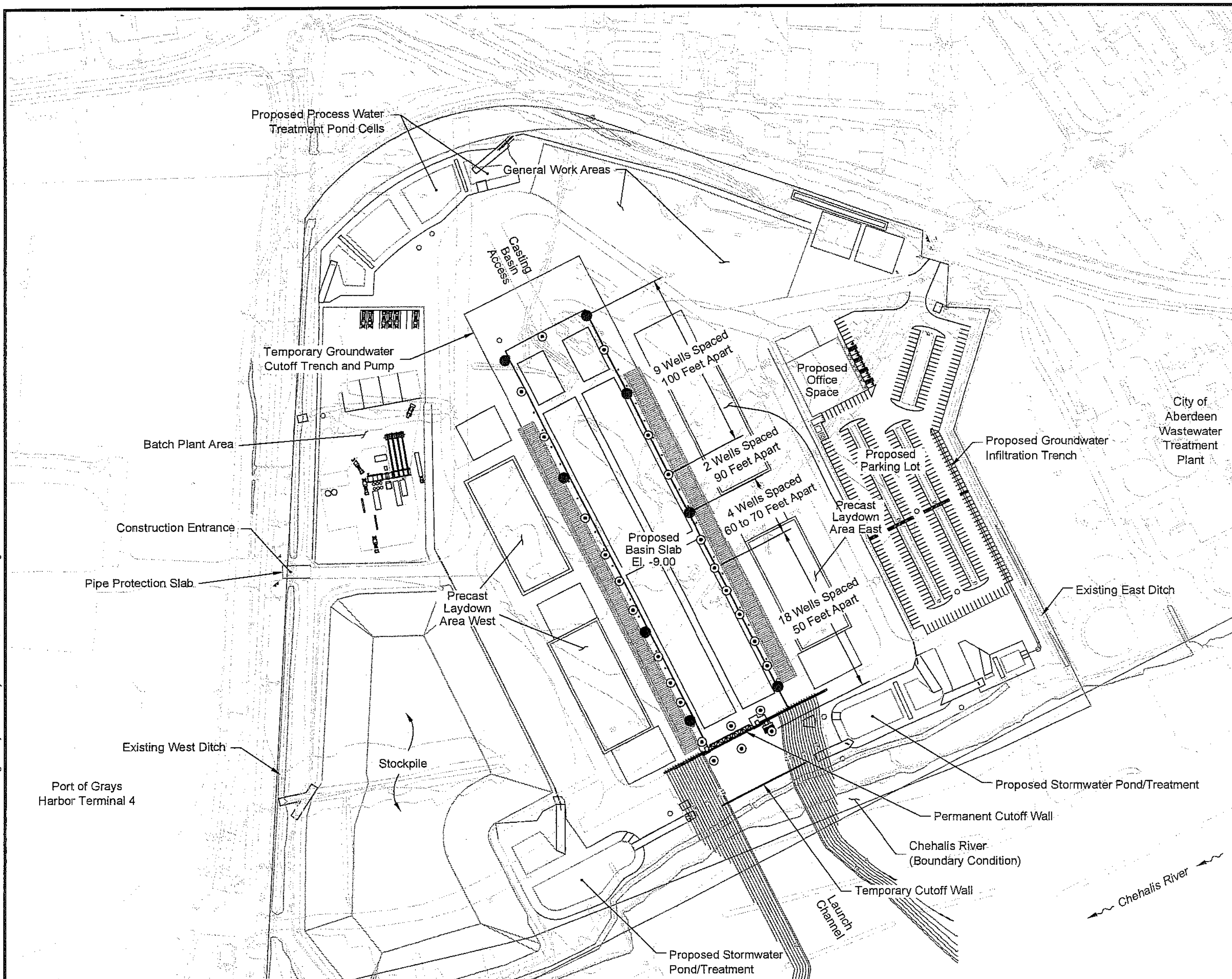
FIG. 36



NOTE

This figure uses basemap data from drawing file *49854_GEO_s_001.dwg* and *XL2672_AL_BP_Base Map-Default.dwg* , received 7-26-10 from HNTB.

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DEWATERING WELL LAYOUT PLAN

December 2010 | 21-1-21190-015

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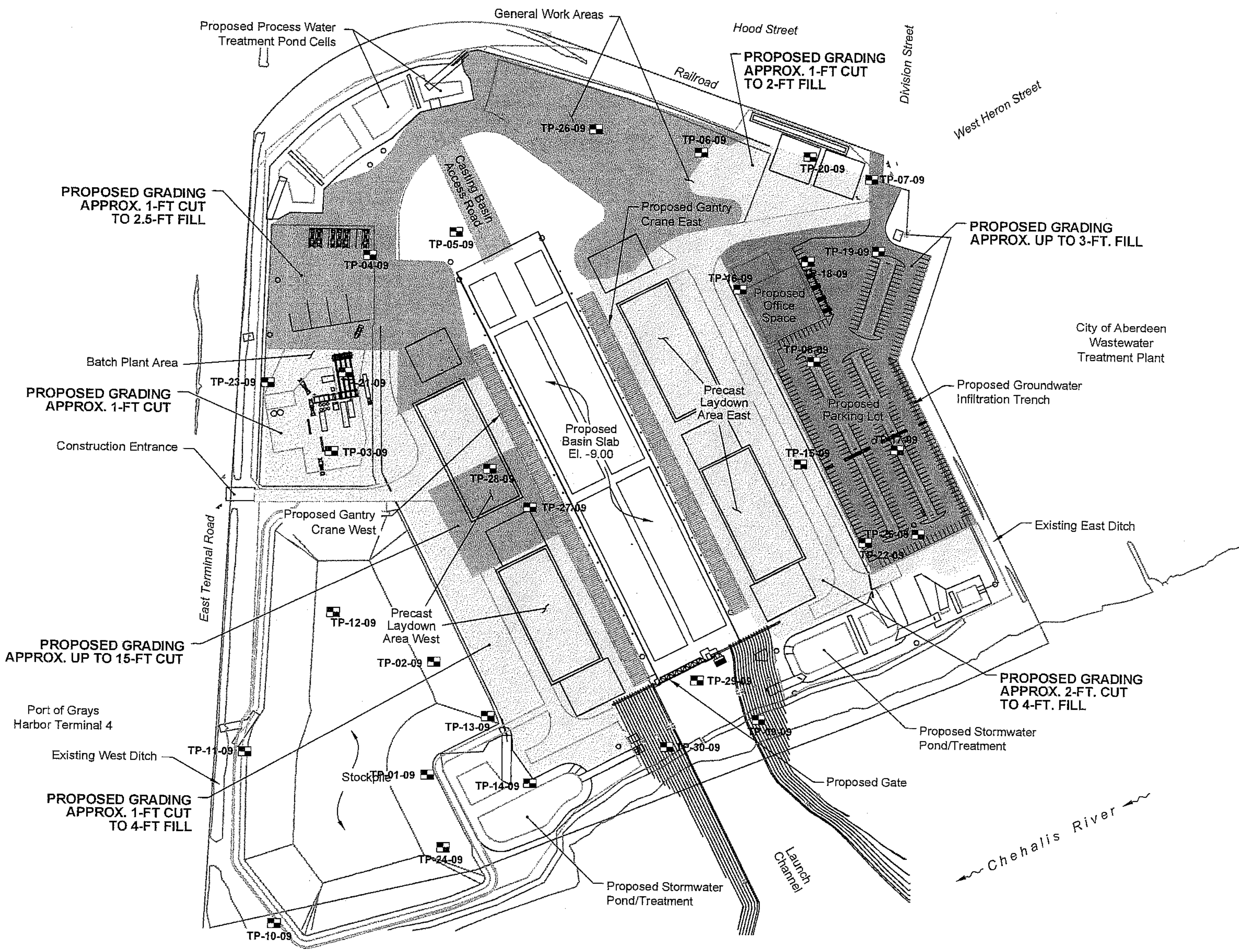
FIG. 37

Filename: J:\211\21190-014\21-1-21190-014 Site.dwg Layout: Well Layout Date: 01-12-2011 ' Login: cnt



LEGEND

- TP-02-09 Test Pit Designation and Approximate Location
- Grading Area 1
- Grading Area 2
- Grading Area 3
- Grading Area 4

NOTES

1. This figure is based on *XL2672_AL_BP_Base Map-Default.dwg* and, *Figure 3-10.dwg*, received 5-10, and *49854_GEO_s_001.dgn*, received 7-26-10.
2. Proposed Grading Area 1 (per Table 7, gravel paved area) and Area 3 (per Table 8, asphalt paved area) will provide adequate thickness for subgrade CBR of 10 percent. No geogrid or geotextile placement is required.
3. Proposed Grading Area 2 will require additional excavation to accommodate the proposed gravel surfaced section at a subgrade CBR of 1 percent per Table 7. Geogrid and geotextile requirements are provided in Table 7.
4. The Casting Basin Access Road (Grading Area 4) will be an asphalt paved area with a CBR of 1 percent and asphalt section per Table 8. Geogrid and geotextile requirements are provided in Table 8.
5. Assumes subgrade proof rolled.

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PROPOSED GRAVEL / HMA SURFACED AREAS

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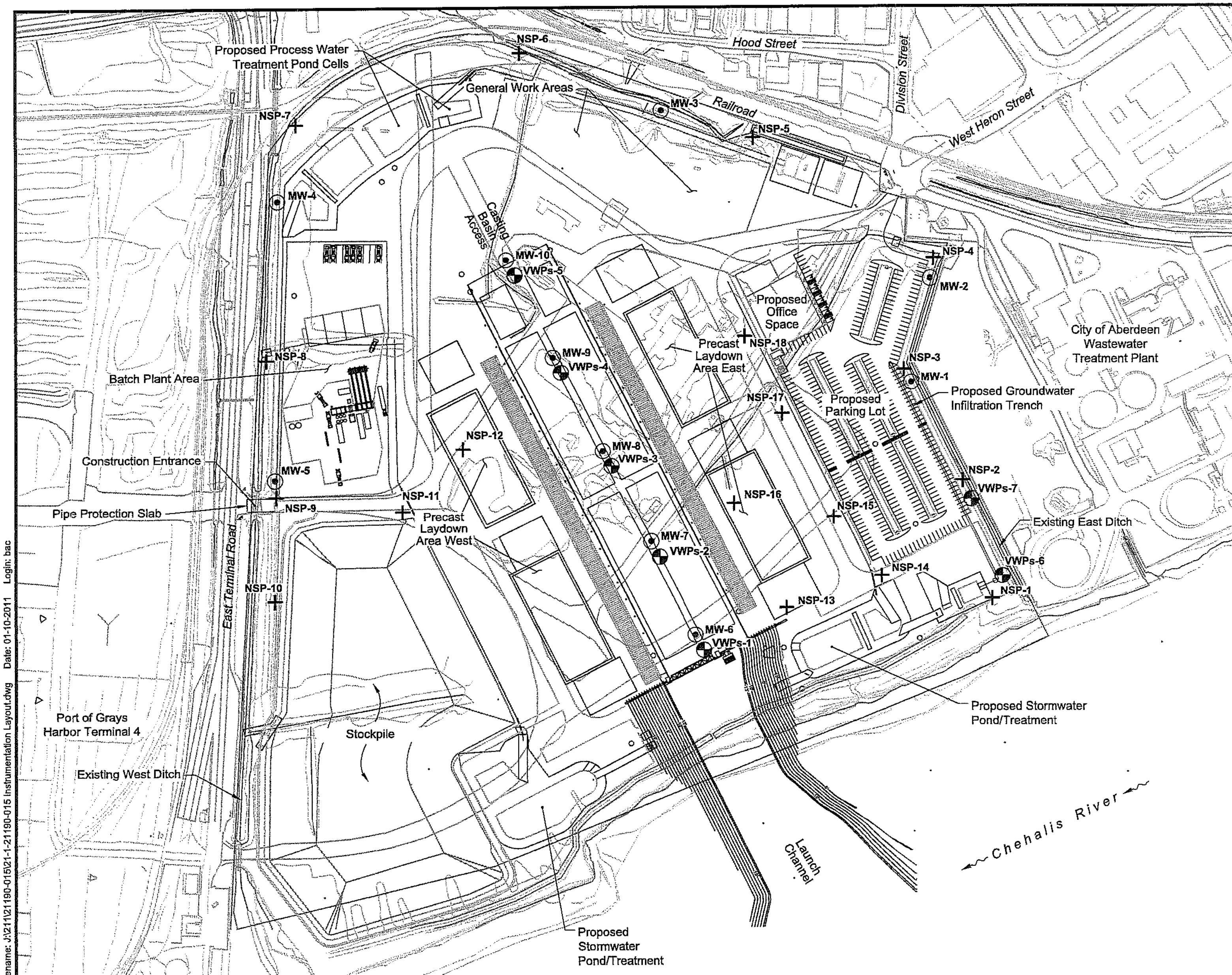
FIG. 38



LEGEND

- VWPs-1 — Proposed Vibrating Wire Piezometers (Shallow and Deep)
- MW-1 — Proposed Groundwater Monitoring Well
- NSP-1 — Proposed Near Surface Settlement Point

NOTES

1. This figure is based on *XL2672_AL_BP_Base Map-Default.dwg* and, *Figure 3-10.dwg*, received 5-10, and *49854_GEO_s_001.dgn*, received 7-26-10.
2. Where practical, install near-surface settlement points on surface electrical vaults and panels.
3. Locations of instrumentation are approximate. Instruments will be located in the field.

0 200 400

Scale in Feet

SR 520 Pontoon Casting Facility
Aberdeen, Washington

GEOTECHNICAL INSTRUMENTATION LAYOUT

December 2010 21-1-21190-015

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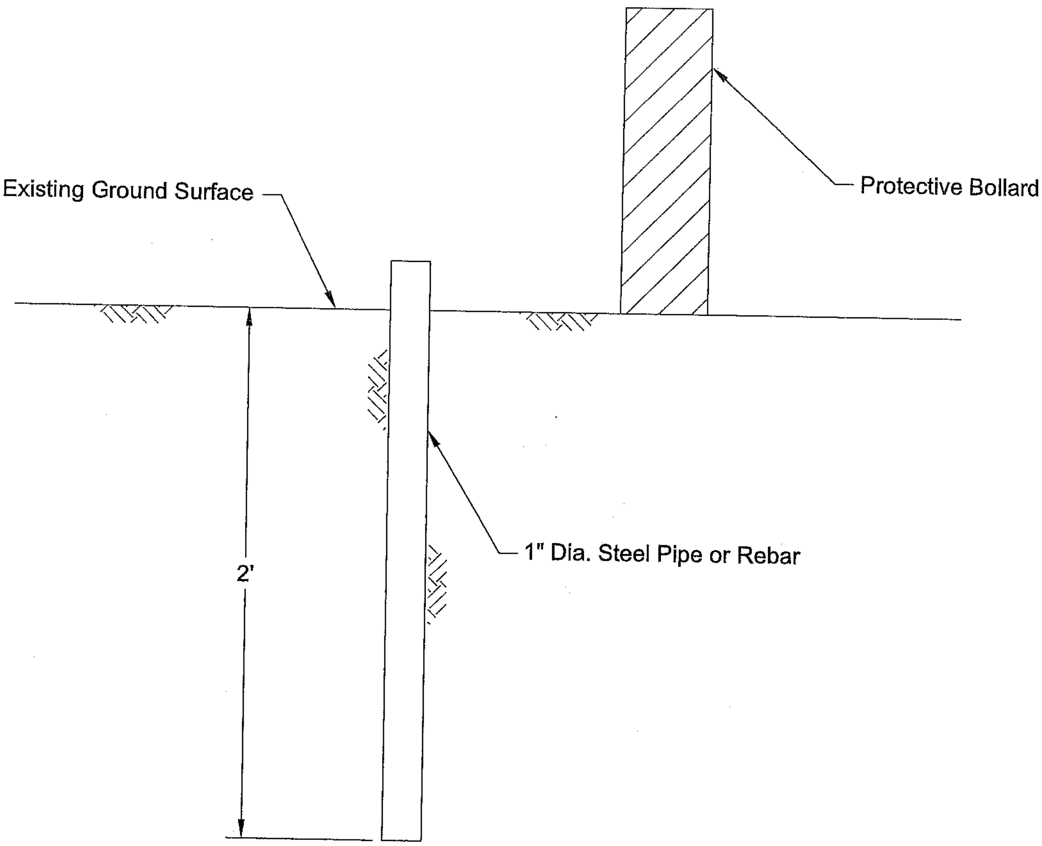
Geotechnical and Environmental Consultants

FIG. 39

Filename: J:\211\21190-015\21-1-21190-015 Instrumentation Layout.dwg Date: 01-10-2011 Login: bac



Filename: F:\21-1-21190 SR 520 Casting Basin\ACAD\21-1-21190 Fig 40 Settlement Point.dwg Date: 01-06-2011 Login: kpc

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Aberdeen, Washington

TYPICAL NEAR SURFACE SETTLEMENT POINT

December 2010 21-1-21190-015

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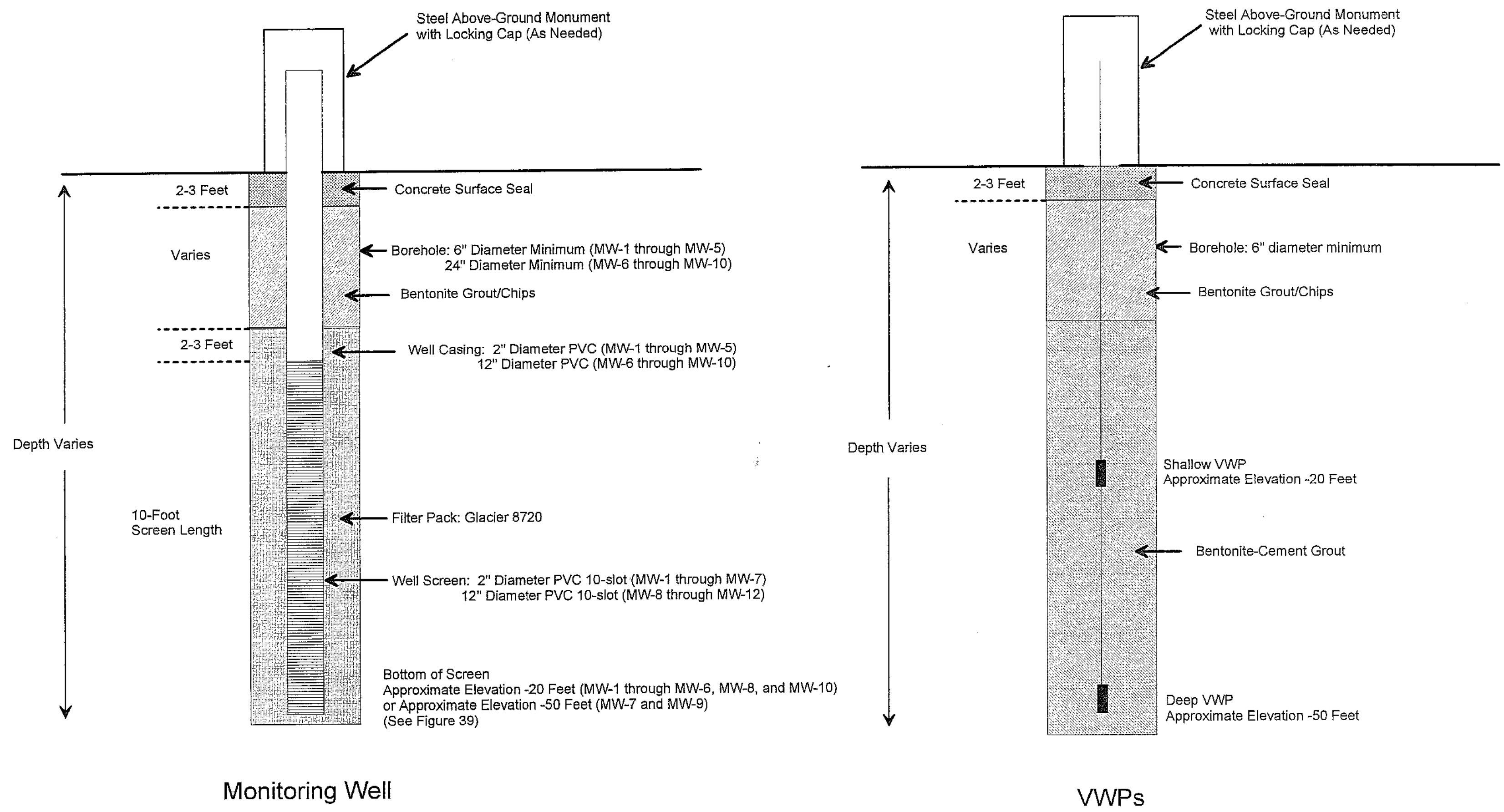
FIG. 40



Not to Scale

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TYPICAL MONITORING WELL AND VWP SCHEMATIC

December 2010 21-1-21190-014

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FIG. 41

APPENDIX A

SHANNON & WILSON, INC.
SUBSURFACE EXPLORATIONS

APPENDIX A

SHANNON & WILSON, INC.
SUBSURFACE EXPLORATIONS

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FIGURES

APPENDIX A

SHANNON & WILSON, INC.
SUBSURFACE EXPLORATIONS

A.1 INTRODUCTION

The subsurface exploration program consisted of drilling and sampling two soil borings, designated BH-1-10 and BH-2-10, completed between March 29 and April 2, 2010. The locations of the field explorations were surveyed and marked by Kiewit-General (KG). The borings were advanced to depths ranging between 195 to 200 feet. The locations of the borings are shown in the Site and Exploration Plan, Figure 2.

A.2 SOIL BORINGS

A.2.1 Drilling Procedures

Gregory Drilling, Inc. drilled the soil borings under subcontract to KG using a truck-mounted CME 85 drill rig and mud rotary techniques. The borings drilled using a 6¼-inch inside-diameter continuous flight hollow-stem auger to a depth of 25 and 20 feet for borings BH-1-10 and BH-2-10, respectively. Below these depths the remaining length of the boring was drilled using mud-rotary techniques. The mud-rotary method consists of drilling the subsurface soils and removing the cuttings by circulation of a bentonite/water mix drilling mud. A settling tank at the ground surface collected the cuttings while the mud was recirculated into the boring. Gregory Drilling, Inc. placed the drill cuttings in barrels that were later cleaned out by a vacuum truck.

Field screening was performed to evaluate for the presence of contamination. Field screening included visual and olfactory observations of the soil samples obtained above and below the groundwater level. Based on visual and olfactory methods of observation, no signs of potential contamination were identified in the boreholes.

A.2.2 Soil Sampling

Disturbed samples from the boring were obtained in conjunction with the Standard Penetration Test (SPT). SPTs were performed in general accordance with the ASTM International (ASTM) Designation: D 1586, generally at 5-foot intervals. This test consists of driving a 2-inch outside-diameter (O.D.), split-spoon sampler a total distance of 18 inches into the bottom of the boring with a 140-pound hammer falling 30 inches. The number of blows required to cause the last 12 inches of penetration is termed the Standard Penetration Resistance, or blow count. When penetration resistances exceeded 50 blows for 6 inches or less of penetration, the test was terminated. The penetration resistances were recorded by our field representative and are plotted on the boring logs. These values provide a means by which to evaluate the relative density or compactness of cohesionless (granular) soils and the consistency (stiffness) of cohesive soils as described in Figure A-1.

The split-spoon sampler used during the penetration testing recovers a disturbed sample of the soil, which is useful for identification purposes. The samples were sealed in jars and returned to our Seattle, Washington, laboratory for testing.

At selected locations, relatively undisturbed samples were obtained in general accordance with ASTM Designation: D 1587-00, Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils. This sampling method employs a thin-walled, steel tube connected to a sampling head attached to the drill rods. The 3-inch O.D. tube is pushed by the hydraulic rams of the drill rig into the bottom of the borehole for a distance of 2 feet. The tube is then retracted to obtain the sample and the top and bottom of the sampling tube are sealed with plastic caps and tape to preserve the field moisture conditions. The sample tubes were then stored upright and returned to our Seattle, Washington, laboratory.

A.2.3 Soil Classification

A representative from Shannon & Wilson, Inc. was present throughout the field exploration to observe the drilling and sampling operations, retrieve representative soil samples for subsequent laboratory testing, and prepare descriptive field logs of the explorations. Boring sample classifications were based on ASTM Designation: D 2487-98, Standard Test Method for Classification of Soil for Engineering Purposes, and ASTM Designation: D 2488-93, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). The Unified Soil Classification System (USCS), as described in Figure A-1 of this appendix, was used to classify the material encountered.

A.2.4 Geophysical Testing

Geophysical tests were performed by GeoVision Geophysical Services Testing in borings BH-1-10 and BH-2-10. The geophysical tests results are presented in Appendix C.

A.2.5 Boring Logs

The Shannon & Wilson boring logs are presented in Figures A-2 and A-3. A boring log is a written record of the subsurface conditions encountered. It graphically illustrates the geologic units (layers) encountered in the boring and the USCS symbol of each geologic layer. It also includes the blow count and natural water content (where tested). Other information shown in the boring logs includes the groundwater-level observations made during drilling, ground surface elevations, types and depths of sampling, and Atterberg Limits (where tested) and the percent by weight of fine grained material passing the #200 sieve (where tested).

A.3 GROUNDWATER OBSERVATIONS

A.3.1 At Time of Drilling

Where observed, groundwater was noted during drilling and is indicated on the boring logs.

A.3.2 Vibrating Wire Piezometers

Two vibrating wire piezometers (VWPs) were installed in borings BH-1-10 and BH-2-10 to measure groundwater levels. The VWPs were installed at 28 and 107 feet below the ground surface (bgs) in boring BH-1-10 and at 45 and 70 feet bgs in boring BH-2-10.

Each VWP consists of a vibrating wire pressure transducer contained in a stainless steel housing. Water pressure acts against a low-air-entry filter at one end of the housing. The transducer is connected to a signal cable that is routed up the borehole to the ground surface. Each VWP was lowered to a specified depth below the ground surface and grouted into place. A data logger was connected to the signal cable of each VWP to collect data readings at select intervals for long-term groundwater level monitoring. Each data reading is compared with calibrations and measurements that were performed before installation. The measured values and calibration information were then used to calculate water pressure acting on the VWP. All VWPs used were Geokon brand with 350- or 700-kilopascal pressure ranges. VWP installation depths and groundwater levels are shown on the boring logs.

A.4 REFERENCE

ASTM International (ASTM), 2010, Annual book of standards, construction, v. 4.08, soil and rock (I): D 420 – D 5876: West Conshohocken, Pa., ASTM International.

Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D 2488-93) unless otherwise noted.

S&W CLASSIFICATION OF SOIL CONSTITUENTS

- MAJOR constituents compose more than 50 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace of gravel).

MOISTURE CONTENT DEFINITIONS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

ABBREVIATIONS

ATD	At Time of Drilling
Elev.	Elevation
ft	feet
FeO	Iron Oxide
MgO	Magnesium Oxide
HSA	Hollow Stem Auger
ID	Inside Diameter
in	inches
lbs	pounds
Mon.	Monument cover
N	Blows for last two 6-inch increments
NA	Not applicable or not available
NP	Non plastic
OD	Outside diameter
OVA	Organic vapor analyzer
PID	Photo-ionization detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split spoon sampler
SPT	Standard penetration test
USC	Unified soil classification
WOH	Weight of hammer
WOR	Weight of drill rods
WLI	Water level indicator

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND* - Fine	#200 to #40 (0.08 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL* - Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Bent. Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Bedrock
PVC Screen	
Vibrating Wire	

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SOIL CLASSIFICATION AND LOG KEY

June 2010 21-1-21190-015

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FIG. A-1
Sheet 1 of 2

BORING CLASS1 21-21190.GPJ SWNEW.GDT 5/6/10

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From USACE Tech Memo 3-357)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravels *(less than 5% fines)*	GW	Well-graded gravels, gravels, gravel/sand mixtures, little or no fines.
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with Fines *(more than 12% fines)*	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sands *(less than 5% fines)*	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly graded sand, gravelly sands, little or no fines
		Sands with Fines *(more than 12% fines)*	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Inorganic silts of low to medium plasticity, rock flour, sandy silts, gravelly silts, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic	OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt
			CH	Inorganic clays of medium to high plasticity, sandy fat clay, or gravelly fat clay
		Organic	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTE: No. 4 size = 5 mm; No. 200 size = 0.075 mm

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND)* indicate that the soil may fall into one of two possible basic groups.

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SOIL CLASSIFICATION AND LOG KEY

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FIG. A-1
Sheet 2 of 2

BORING_CLASS2 21-21190.GPJ SWNEW.GDT 5/6/10

Total Depth:	200 ft.	Northing:		Drilling Method:	Mud Rotary	Hole Diam.:	6 in.
Top Elevation:	~	Easting:		Drilling Company:	Gregory	Rod Diam.:	2.5"
Vert. Datum:		Station:		Drill Rig Equipment:	CME 85	Hammer Type:	Automatic
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

SOIL DESCRIPTION	Depth, ft.
Very loose, brown, gravelly SAND, numerous wood fragments; Fill.	
Very soft, gray, slightly fine sandy, slightly clayey SILT; moist; scattered wood fragments; ML, MH.	12.0
Very loose, gray, slightly silty SAND; moist; SP-SM.	31.0
Very soft, gray, clayey SILT; moist; numerous roots; MH.	38.0
Very soft, gray, sandy SILT interbedded with very loose, silty SAND; moist; trace to abundant wood fragments and roots; ML, SM.	42.0
Loose, dark gray, silty SAND; moist; SM.	86.0
Medium stiff, gray, sandy SILT interbedded with loose, silty SAND; moist; ML, SM.	88.0
Loose, dark gray, silty SAND; moist; occasional silt lamination; SM. - 1-inch wood seam at 96 feet.	91.0
	100.0
	103.0

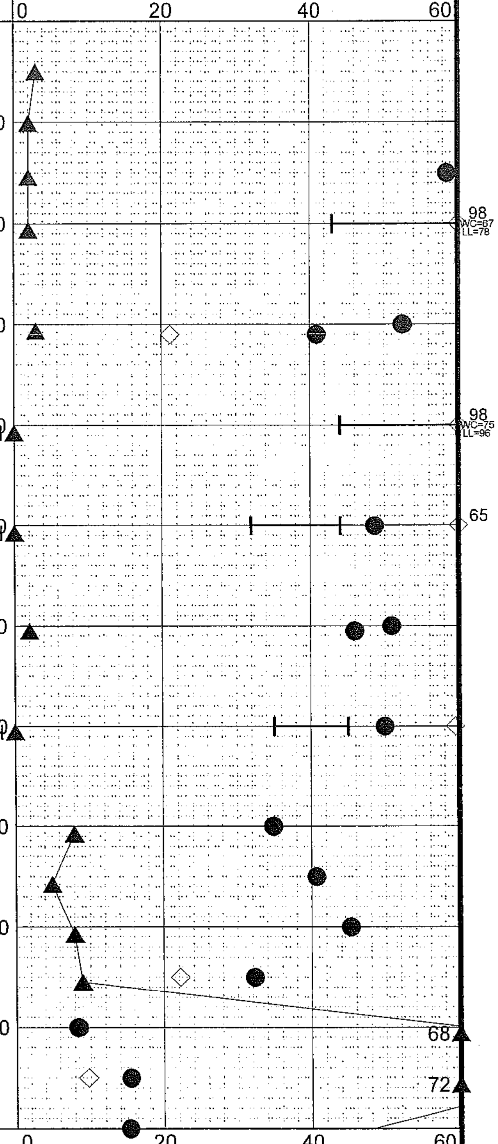
Symbol | Samples | Ground Water | Depth, ft.

Samples: 1 *, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15, 16, 17, 18, 19, 20, 21

Ground Water: 5/11/2010 (1), 5/11/2010 (2)

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches



CONTINUED NEXT SHEET

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- 3.0" O.D. Osterberg Sample
- 2.5" O.D. Split Spoon Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout
- Ground Water Level in VWP

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├—●—┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

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LOG OF BORING BH-1-10

August 2010 21-1-21190-015

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FIG. A-2
Sheet 1 of 2

MASTER LOG E 21-21190.GPJ SHAN WIL.GDT 1/5/11 Log: CIJ Rev: RAM Typ: CLP

REV 2

Total Depth:	200 ft.	Northing:		Drilling Method:	Mud Rotary
Top Elevation:	~	Easting:		Drilling Company:	Gregory
Vert. Datum:		Station:		Drill Rig Equipment:	CME 85
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.: 6 in.
Rod Diam.: 2.5"
Hammer Type: Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Very dense, gray, silty, sandy GRAVEL; moist; GM.

Very dense, gray, silty, gravelly SAND; moist to wet; SW-SM.
- Scattered wood fragments at 111 feet.

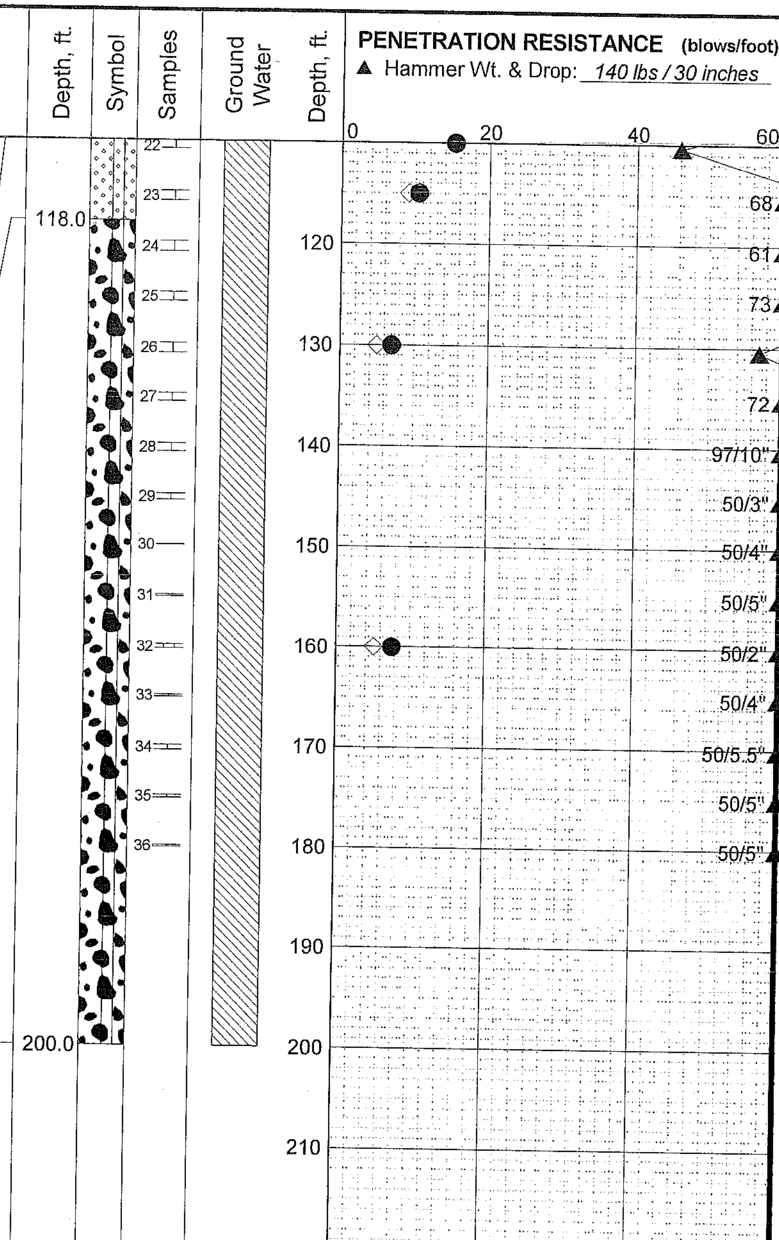
118.0

Very dense, gray, clean to silty, sandy GRAVEL; wet; GW-GM, GP, GP-GM, GM.

Not sampled below 185 feet depth. Soil description based on drill action. Boring extended to 200 feet depth for geophysical testing.

200.0

BOTTOM OF BORING
COMPLETED 3/31/2010



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- 3.0" O.D. Osterberg Sample
- 2.5" O.D. Split Spoon Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout
- Ground Water Level in VWP

◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES
1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

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LOG OF BORING BH-1-10

August 2010 21-1-21190-015

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FIG. A-2
Sheet 2 of 2

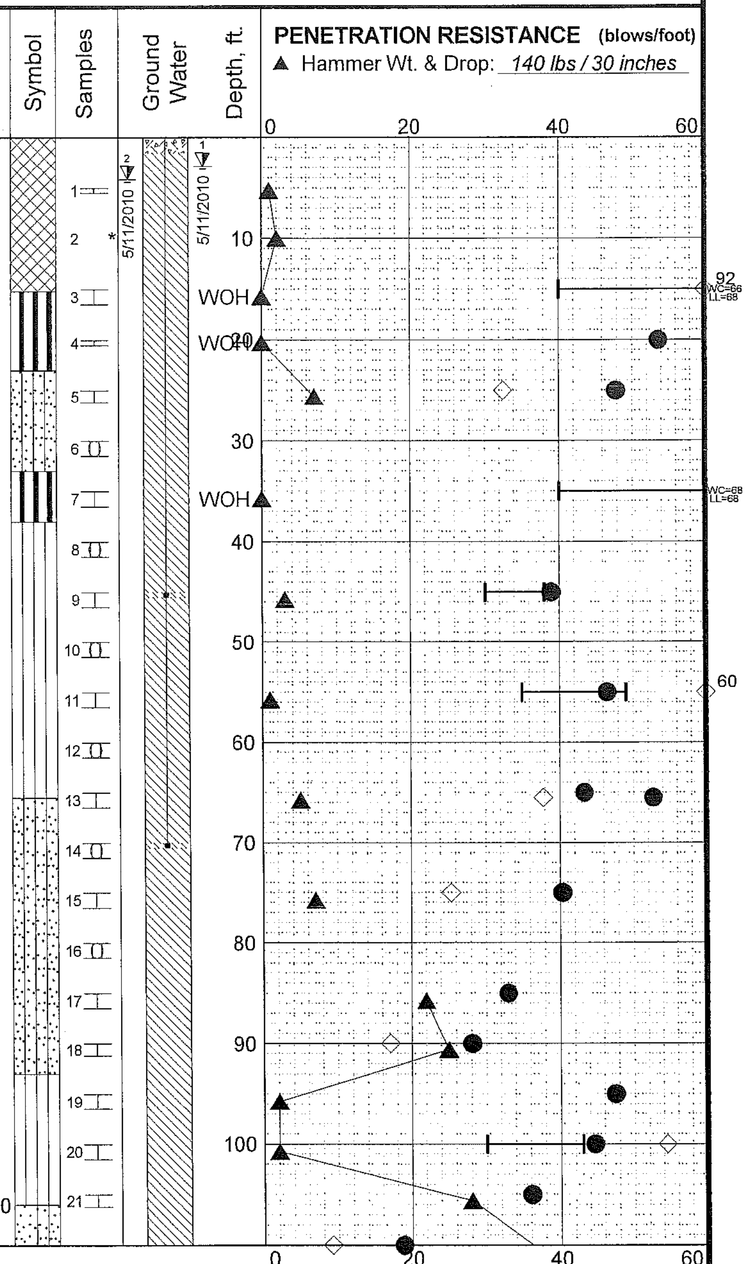
REV 2

Total Depth:	*200 ft.*	Northing:		Drilling Method:	*Mud Rotary*	Hole Diam.:	*5.5 in.*
Top Elevation:	~	Easting:		Drilling Company:	*Gregory*	Rod Diam.:	*2.5"*
Vert. Datum:		Station:		Drill Rig Equipment:	*CME 85*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:		Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.
Very loose, brown, sandy GRAVEL; moist; numerous wood fragments; moist; Fill.	15.2
Very soft, gray, slightly fine sandy, slightly clayey SILT; moist; scattered roots; MH.	23.0
Loose, gray, slightly clayey, silty SAND interbedded with medium stiff, fine sandy SILT; moist; scattered wood fragments; SM, ML..	33.0
Very soft, gray, clayey SILT; moist; numerous roots; MH.	38.0
Very soft to medium stiff, gray, fine sandy SILT to silty, fine SAND; moist; scattered wood fragments; ML, SM.	65.5
Loose to medium dense, slightly silty to silty SAND; moist; occasional wood fragments, occasional silt laminations; SM, SP-SM, SC.	93.0
Very loose to loose, gray, sandy SILT interbedded with very loose, silty SAND and ; moist; ML, SM.	106.0



CONTINUED NEXT SHEET

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- 3.0" O.D. Osterberg Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout
- Ground Water Level in VWP

◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

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LOG OF BORING BH-2-10

August 2010 21-1-21190-015

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FIG. A-3
Sheet 1 of 2

MASTER LOG E 21-21190.GPJ SHAN WIL.GDT 1/5/11 Log: CJJ Rev: RAM Typ: CLP

REV 2

Total Depth:	200 ft.	Northing:		Drilling Method:	Mud Rotary
Top Elevation:	~	Easting:		Drilling Company:	Gregory
Vert. Datum:		Station:		Drill Rig Equipment:	CME 85
Horiz. Datum:		Offset:		Other Comments:	

Hole Diam.: 5.5 in.
Rod Diam.: 2.5"
Hammer Type: Automatic

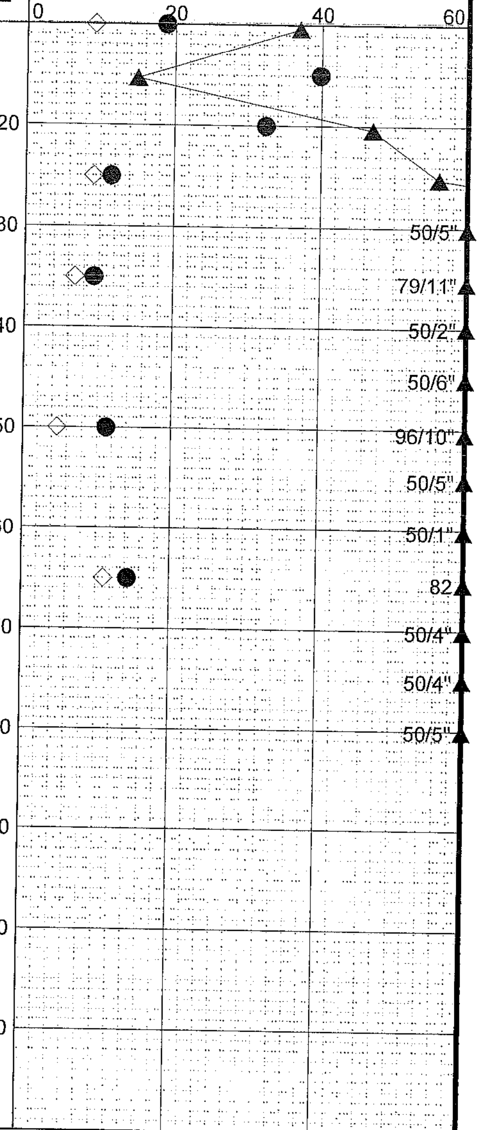
SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

SOIL DESCRIPTION	Depth, ft.	Samples
Medium dense to dense, slightly silty SAND; moist; occasional silt lamination; SP-SM.	115.2	22, 23
Stiff, gray, silty CLAY; moist; CL.	118.0	
Dense to very dense, slightly silty to silty, trace to gravelly SAND; moist; SW-SM, SW-SC.	138.0	24, 25, 26, 27
Very dense, gray, silty, sandy GRAVEL; wet; GM.	163.0	28 *, 29, 30, 31, 32
Very dense, dark gray, slightly clayey, slightly silty, slightly gravelly SAND; wet; SW-SC.	168.0	33
Very dense, gray, silty, sandy GRAVEL; wet; GM. Not sampled below 185 feet depth. Soil description based on drill action. Boring extended to 200 feet depth for geophysical testing.	200.0	34, 35, 36
BOTTOM OF BORING COMPLETED 1/1/2010		

Symbol | Ground Water | Depth, ft.

PENETRATION RESISTANCE (blows/foot)
▲ Hammer Wt. & Drop: 140 lbs / 30 inches



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- 3.0" O.D. Osterberg Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout
- Ground Water Level in VWP

◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. Groundwater level, if indicated above, is for the date specified and may vary.
3. USCS designation is based on visual-manual classification and selected lab testing.

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LOG OF BORING BH-2-10

August 2010 | 21-1-21190-015

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FIG. A-3
Sheet 2 of 2

REV 2

MASTER_LOG_E 21-21190.GPJ SHAN_WIL.GDT 1/5/11 Log: CJJ Rev: RAM Typ: CLP

APPENDIX B

SHANNON & WILSON, INC.
GEOTECHNICAL LABORATORY TESTING

APPENDIX B

SHANNON & WILSON, INC.
GEOTECHNICAL LABORATORY TESTING

TABLE OF CONTENTS

FIGURES

APPENDIX B

SHANNON & WILSON, INC.
GEOTECHNICAL LABORATORY TESTING

B.1 INTRODUCTION

Samples collected from the two borings (BH-1-10 and BH-2-10) during the explorations completed between March 29 and April 2, 2010, were sealed in jars and tubes and returned to our Seattle, Washington, laboratory for testing. Selected disturbed samples were tested to determine the basic index properties and the engineering characteristics of the subsurface soils at the site. Tests were conducted in general accordance with applicable ASTM International (ASTM) standards. The results of these tests are presented in Appendix D of this report.

B.2 VISUAL CLASSIFICATION

Each of the soil samples recovered from the borings were visually reclassified in our laboratory using a system based on the ASTM Designation: D 2487, Standard Practice for Classification of Soils for Engineering Purposes, and ASTM Designation: D 2488, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). These ASTM standards use the Unified Soil Classification System (USCS), described in Figure A-1. The individual sample classifications have been incorporated into our boring logs shown in Figures A-2 and A-3.

B.3 INDEX TESTS

B.3.1 Water Content Determination

The natural water content of select soil samples recovered from the field explorations was determined in general accordance with ASTM Designation D 2216, Standard Method of Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass. Comparison of water content of a soil with its index properties can be useful in characterizing soil unit weight, consistency, compressibility, and strength. Water content, where tested, is plotted on each of the boring logs presented in Appendix A.

B.3.2 Grain Size Distribution Analyses

Grain size distribution analyses were performed on 22 samples in general accordance with ASTM Designation: D 422, Standard Method for Particle-Size Analysis of Soils or D 1140, Standard Test Methods for Amount of Material in Soils Finer than No. 200 (75-microgram) Sieve. The general procedures to determine the grain size distribution of a soil sample include sieve analysis, hydrometer analysis, combined analysis, and percentage of fines passing the No. 200 sieve.

Grain size distributions are used to assist in classifying soils and to provide correlation with soil properties, including permeability, behavior when excavated, capillary action, and sensitivity to moisture. Results of the grain size analyses are shown on the appropriate boring logs in Appendix A and on grain size distribution curves shown in Figures B-1 and B-2. Along with each grain size distribution is a tabulated summary containing the group symbol according to the USCS, the sample description, percentage of fines passing the No. 200 sieve, and the natural water content.

B.3.3 Atterberg Limit Determinations

Atterberg Limits were determined on nine samples of fine-grained soil in general accordance with ASTM Designation: D 4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The Atterberg Limits include Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI=LL-PL). They are generally used to assist in classification of soil, to indicate soil consistency (when compared with natural water content), and to provide correlation to soil properties including compressibility and strength.

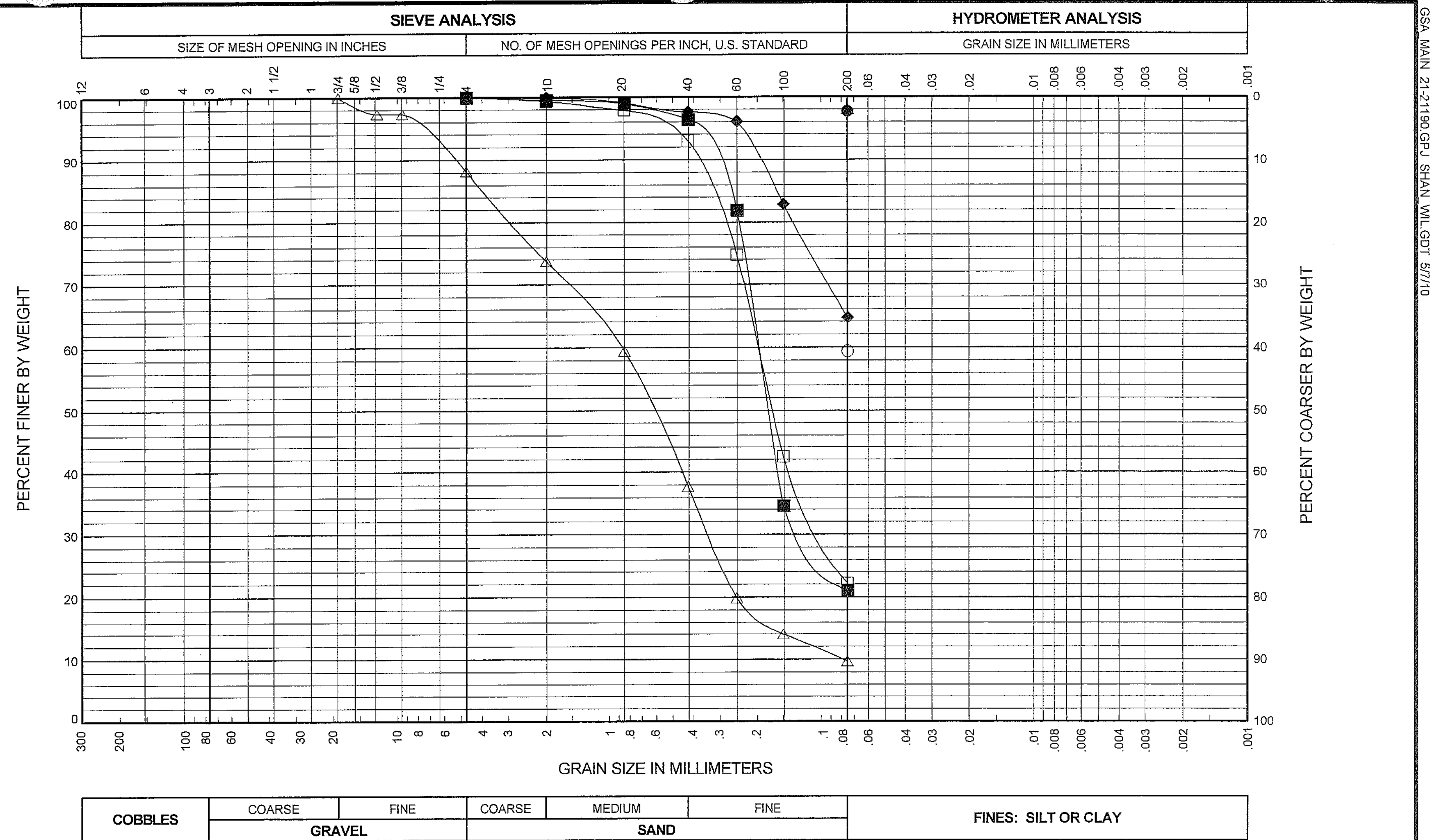
The results of the Atterberg Limits determinations are shown on the appropriate boring logs in Appendix A and in the plasticity charts shown in Figures B-3 and B-4.

B.3.4 Organic Content

Organic contents were determined on two soil samples in general accordance with ASTM Designation: D 2974, Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils. Organic content is generally used to assist in classification of soils, peat or other organic soil.

B.4 REFERENCE

ASTM International (ASTM), 2010, Annual book of standards, construction, v. 4.08, soil and rock (I): D 420 – D 5876: West Conshohocken, Pa., ASTM International.



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● BH-1-10, S-4	20.0	MH	Light brown, slightly clayey SILT, trace of sand; trace of organics	97.6	67.1	78	43	35
■ BH-1-10, S-6	31.0	SM	Brown, silty, fine SAND; trace of organics	21.1	40.9			
▲ BH-1-10, S-8	40.0	MH	Brown, slightly clayey SILT, trace of sand; scattered organics	98.0	75.2	96	44	52
◆ BH-1-10, S-10	50.0	ML	Brown, fine sandy, clayey SILT	64.6	48.6	44	32	12
○ BH-1-10, S-14	70.0	ML	Dark brown, sandy SILT, trace of clay; trace of organics	59.4	49.9	45	35	10
□ BH-1-10, S-19	95.0	SM	Dark gray, silty, fine to medium SAND	22.2	32.3			
△ BH-1-10, S-21	105.0	SW-SM	Dark gray, slightly silty, slightly fine gravelly SAND	9.7	15.5			

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GRAIN SIZE DISTRIBUTION
BORING BH-1-10

June 2010 21-1-21190-015

SHANNON & WILSON, INC.
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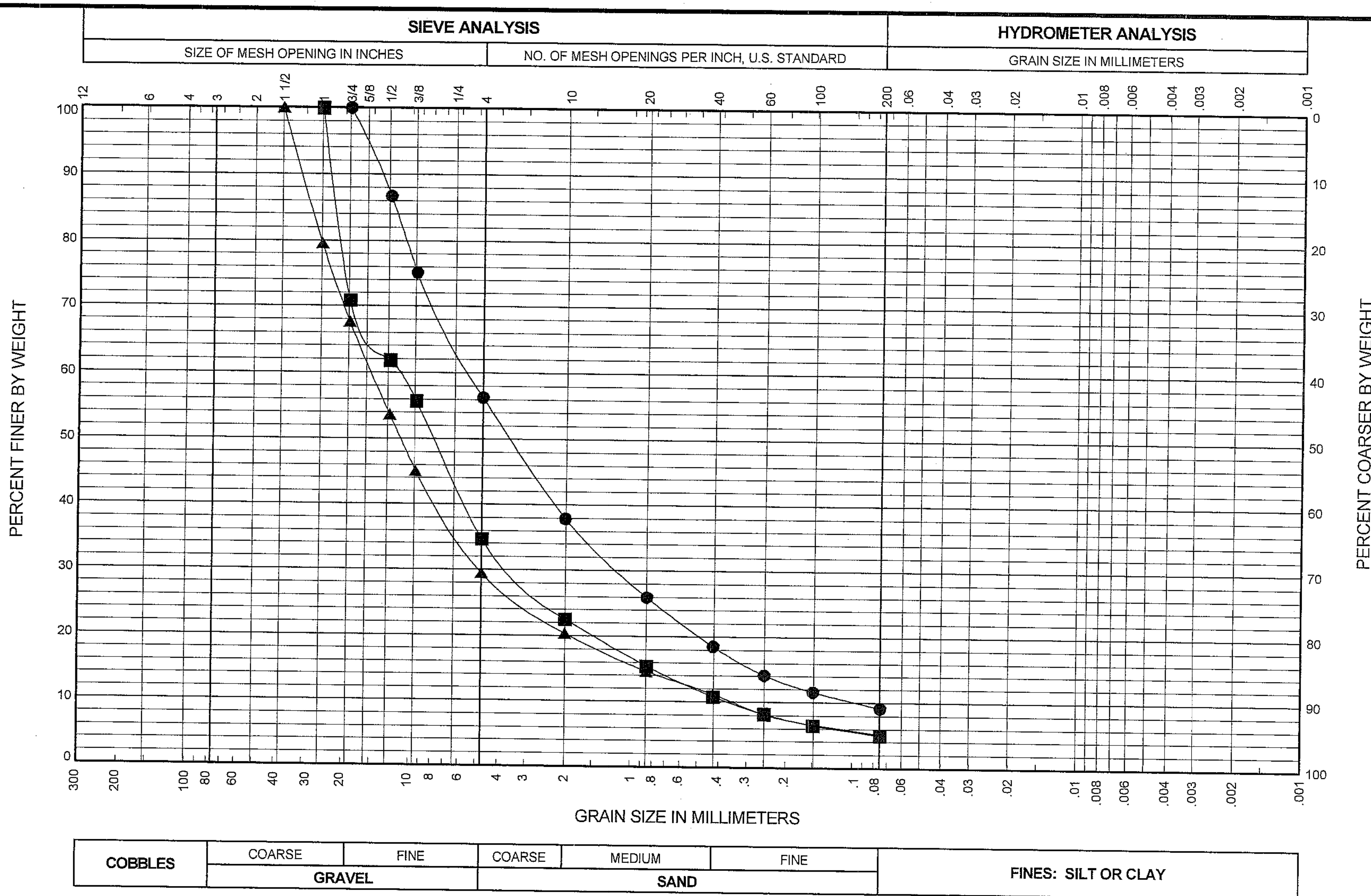
FIG. B-1
Sheet 1 of 2

FIG. B-1



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● BH-1-10, S-23	115.0	SW-SM	Dark gray, slightly silty, fine gravelly SAND	9.1	10.5			
■ BH-1-10, S-26	130.0	GW-GM	Dark gray, slightly silty, sandy GRAVEL	5.0	7.0			
▲ BH-1-10, S-32	160.0	GP-GM	Dark gray, slightly silty, sandy GRAVEL	5.1	7.6			

SR 520 Pontoon Casting Facility
Aberdeen, Washington

GRAIN SIZE DISTRIBUTION
BORING BH-1-10

June 2010

21-1-21190-015

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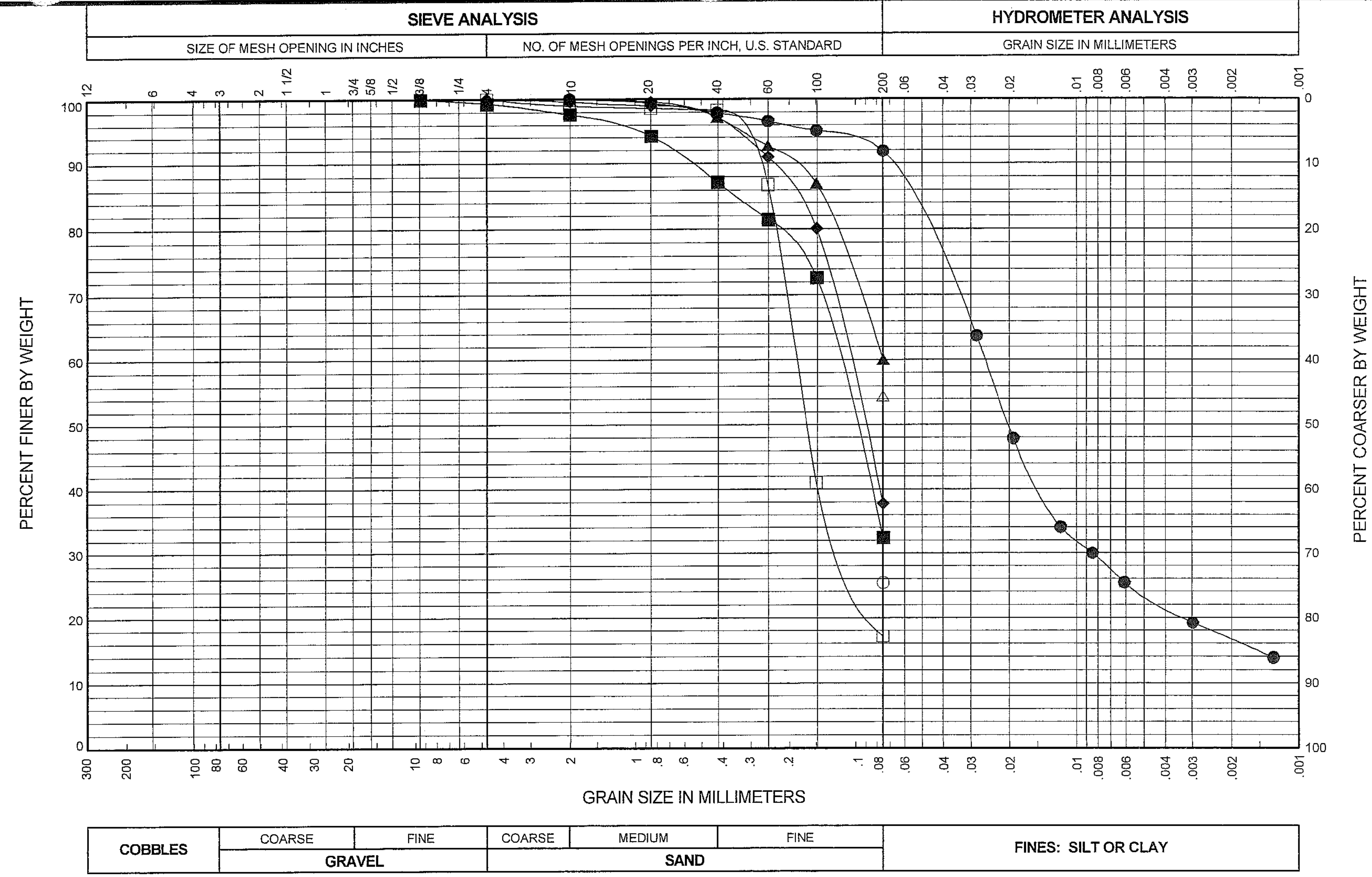
FIG. B-1
Sheet 2 of 2

FIG. B-1



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● BH-2-10, S-3	15.0	MH	Gray, slightly fine to medium sandy, slightly clayey SILT	91.8	65.6	68	40	28
■ BH-2-10, S-5	25.0	SM	Gray, slightly clayey, silty, fine to medium SAND, trace of fine gravel	32.5	47.8			
▲ BH-2-10, S-11	55.0	ML	Gray, fine sandy SILT	60.1	46.5	49	35	14
◆ BH-2-10, S-13	65.5	SM	Gray, slightly clayey, silty, fine SAND	37.8	52.7			
○ BH-2-10, S-15	75.0	SM	Gray, silty, fine SAND	25.4	40.3			
□ BH-2-10, S-18	90.0	SC	Gray, clayey, fine SAND	17.0	28.1			
△ BH-2-10, S-20	100.0	ML	Gray, slightly clayey, fine sandy SILT	54.4	44.6	43	30	13

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GRAIN SIZE DISTRIBUTION BORING BH-2-10

June 2010

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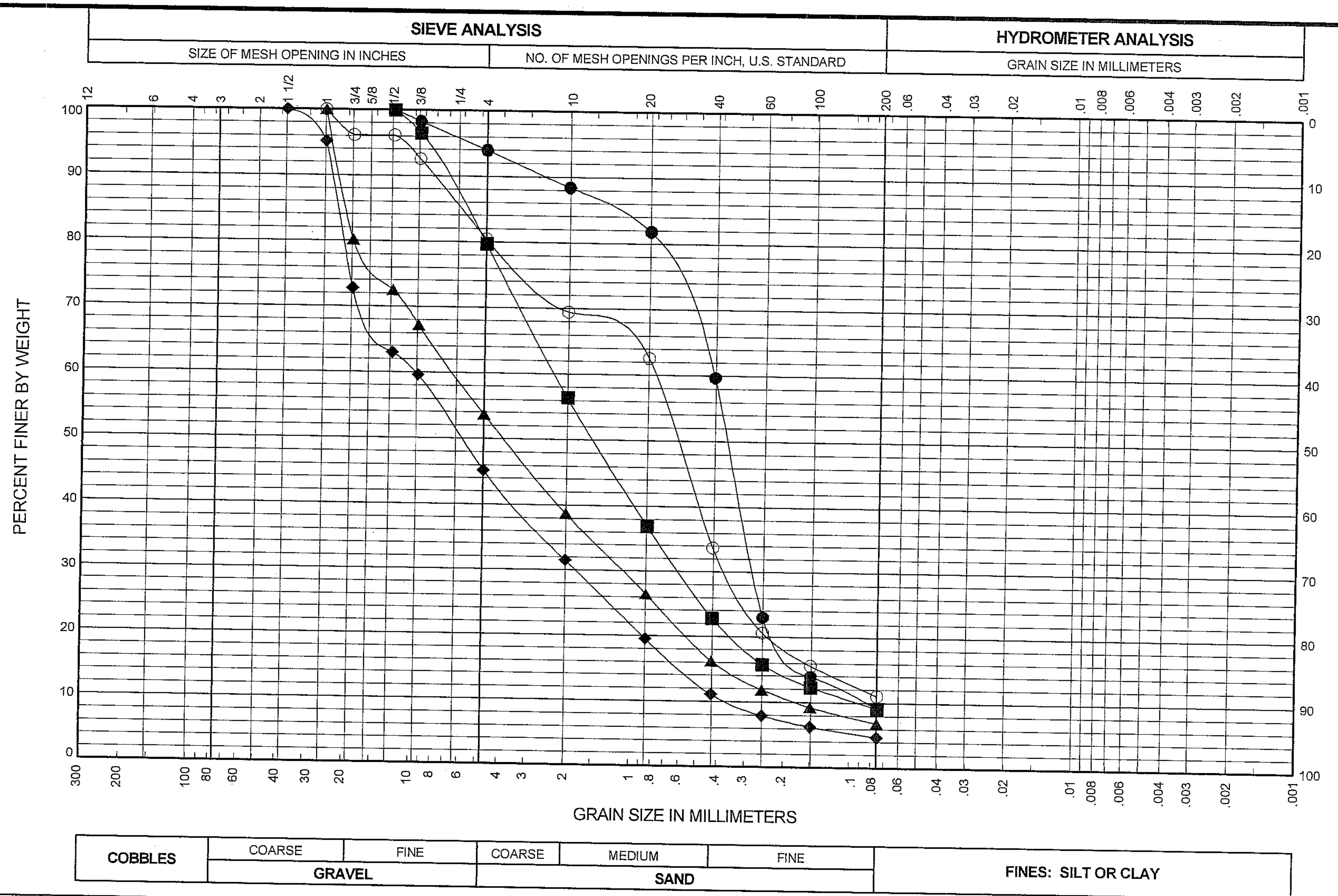
FIG. B-2
Sheet 1 of 2

FIG. B-2



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SAMPLE DESCRIPTION	FINES %	NAT. W.C. %	LL %	PL %	PI %
● BH-2-10, S-22	110.0	SP-SM	Gray, slightly fine gravelly, slightly silty SAND	9.2	18.8			
■ BH-2-10, S-25	125.0	SW-SM	Gray, slightly silty, fine gravelly SAND	9.1	11.5			
▲ BH-2-10, S-27	135.0	SW-SC	Gray, slightly clayey, gravelly SAND	6.9	9.3			
◆ BH-2-10, S-30	150.0	GP	Gray, sandy GRAVEL, trace of clay	4.7	11.2			
○ BH-2-10, S-33	165.0	SW-SC	Gray, slightly clayey, gravelly SAND	11.0	14.3			

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Aberdeen, Washington

GRAIN SIZE DISTRIBUTION
BORING BH-2-10

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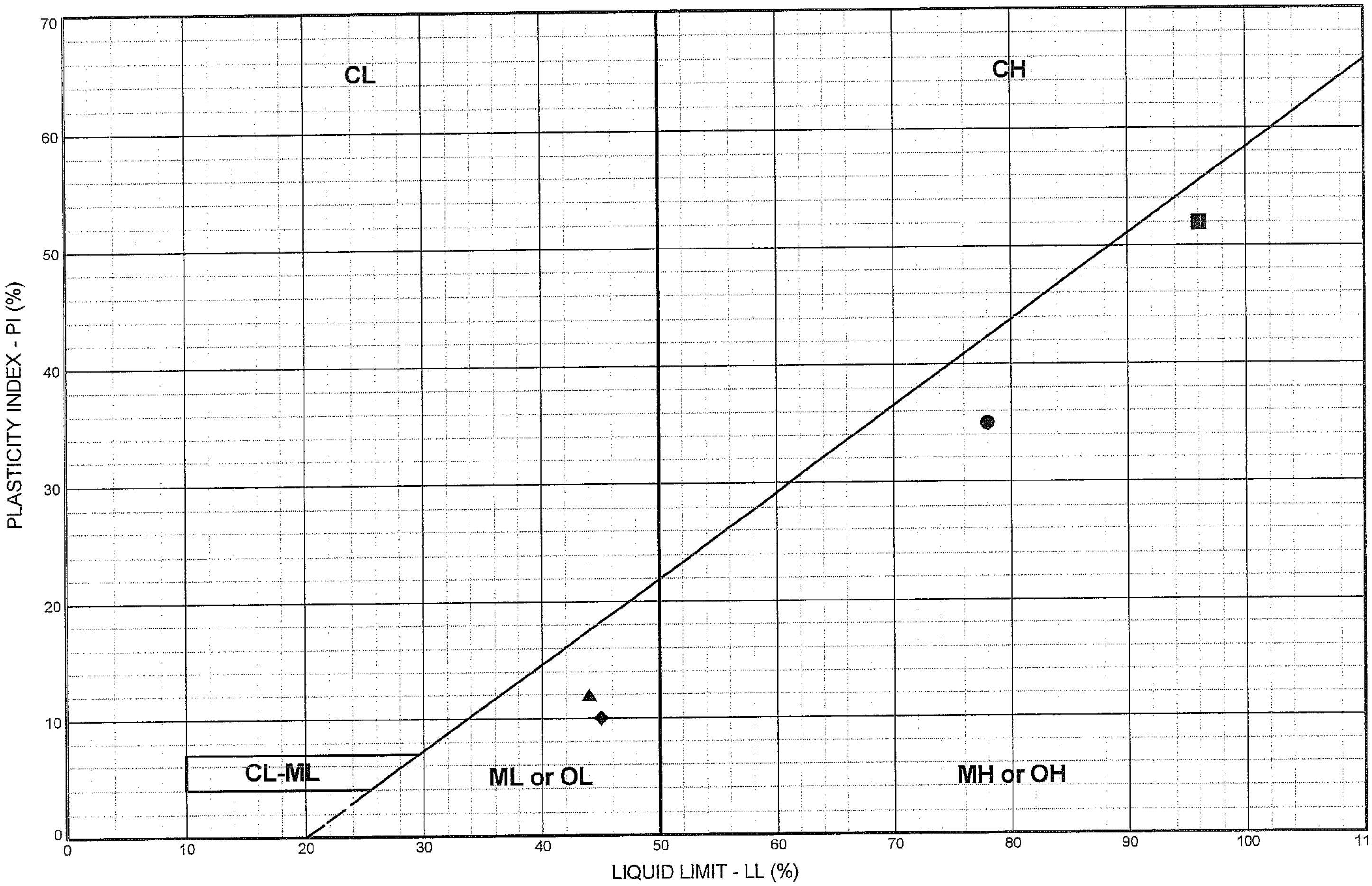
FIG. B-2
Sheet 2 of 2

FIG. B-2



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200, %
● BH-1-10, S-4	20.0	MH	Light brown, slightly clayey SILT, trace of sand; trace of organics	78	43	35	67.1	97.6
■ BH-1-10, S-8	40.0	MH	Brown, slightly clayey SILT, trace of sand; scattered organics	96	44	52	75.2	98.0
▲ BH-1-10, S-10	50.0	ML	Brown, fine sandy, clayey SILT	44	32	12	48.6	64.6
◆ BH-1-10, S-14	70.0	ML	Dark brown, sandy SILT, trace of clay; trace of organics	45	35	10	49.9	59.4

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PLASTICITY CHART
BORING BH-1-10

June 2010 21-1-21190-015

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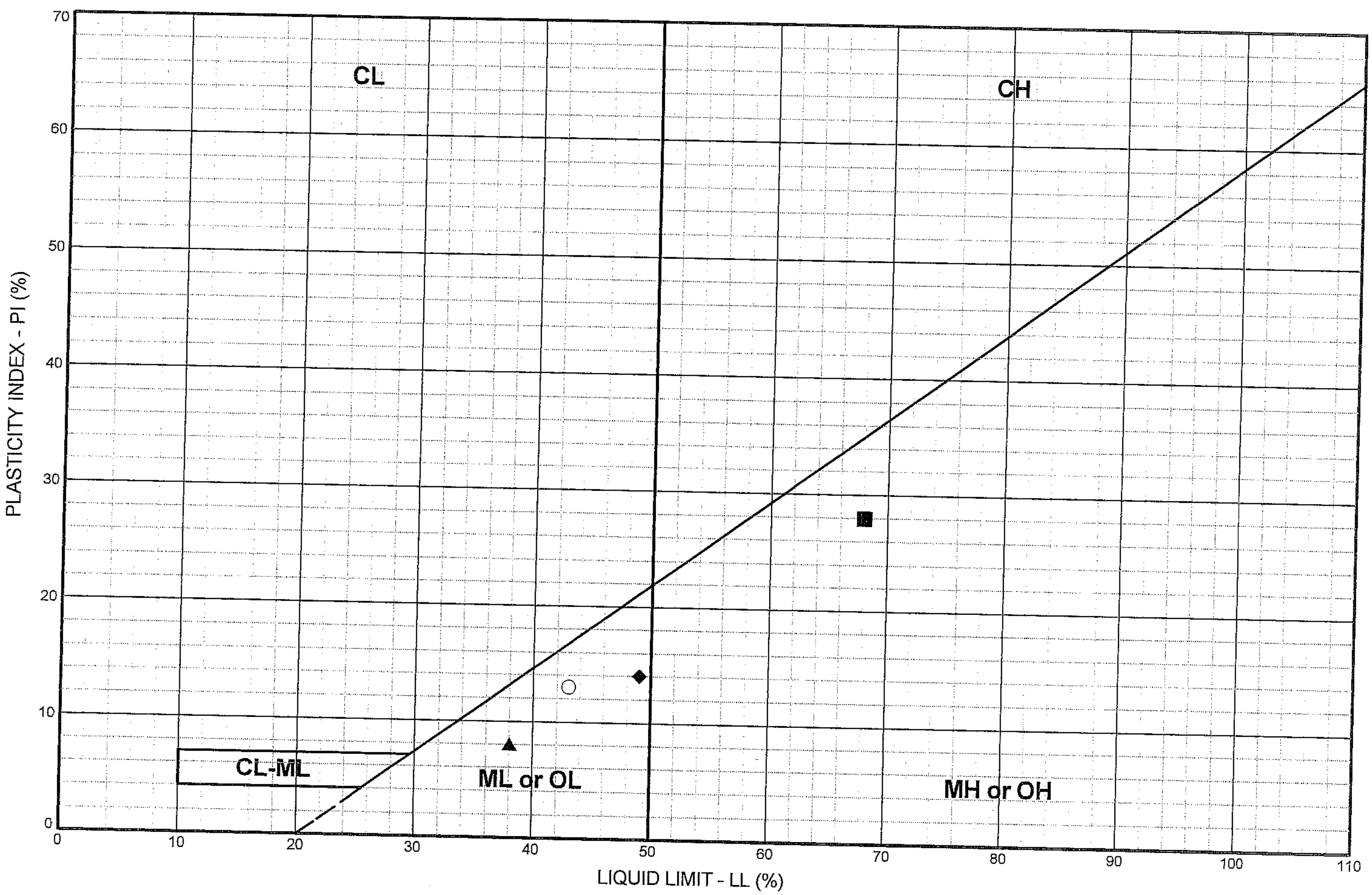
FIG. B-3



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200, %
● BH-2-10, S-3	15.0	MH	Gray, slightly fine to medium sandy, slightly clayey SILT	68	40	28	65.6	91.8
■ BH-2-10, S-7	35.0	MH	Gray, clayey SILT	68	40	28	67.8	
▲ BH-2-10, S-9	45.0	SM	Gray, silty, fine SAND; plasticity likely due to fine organics	38	30	8	38.9	
◆ BH-2-10, S-11	55.0	ML	Gray, fine sandy SILT	49	35	14	46.5	60.1
○ BH-2-10, S-20	100.0	ML	Gray, slightly clayey, fine sandy SILT	43	30	13	44.6	54.4

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PLASTICITY CHART
BORING BH-2-10

June 2010

21-1-21190-015

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FIG. B-4

FIG. B-4

APPENDIX C

GEOVISION FINAL REPORT
520 PONTOON CONSTRUCTION
DESIGN BUILD PROJECT
DATED AUGUST 26, 2010

GEOVision
geophysical services

FINAL REPORT

520 PONTOON CONSTRUCTION DESIGN-BUILD PROJECT

BORING GEOPHYSICS, BORINGS BH-1-10 AND BH-2-10

Report 10107-01 rev 3

August 26, 2010

520 PONTOON CONSTRUCTION DESIGN-BUILD PROJECT

BORING GEOPHYSICS, BORINGS BH-1-10 AND BH-2-10

Report 10107-01 rev 3

August 26, 2010

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APPENDICES

INTRODUCTION

Boring geophysical measurements were collected in 2 uncased borings for the 520 Pontoon Construction Design-Build Project, near Aberdeen, Washington. Geophysical data acquisition was performed on March 31 and April 2, 2010 by Charles Carter of **GEO***Vision*. Data analysis and report preparation was performed by Robert Steller and reviewed by John Diehl of **GEO***Vision*. The work was performed under subcontract with Kewitt General, with Kyle Johnson serving as the point of contact for Kewitt.

This report describes the field measurements, data analysis, and results of this work.

SCOPE OF WORK

This report presents the results of boring geophysical measurements collected on March 31 and April 2, 2010, in 2 uncased borings, as detailed below. The purpose of these studies were to supplement stratigraphic information obtained during Kewitt's soil sampling program and to acquire shear wave velocities and compressional wave velocities as a function of depth.

BORING DESIGNATION	DATES LOGGED	LOCATION		ELEVATION (FEET)
		NORTH	EAST	
BH-1-10	3/31/10	NA	NA	NA
BH-2-10	4/2/10	NA	NA	NA

Elevations shown are referenced to the North American Vertical Datum (NAVD88).

Coordinates are based on the North American Datum (NAD83).

Table 1 Boring logging dates and locations

The OYO Model 170 Suspension Logging Recorder and Suspension Logging Probe were used to obtain in-situ horizontal shear and compressional wave velocity measurements at 1.6-foot intervals. The acquired data were analyzed and a profile of velocity versus depth was produced for both compressional and horizontally polarized shear waves.

A detailed reference for the velocity measurement techniques used in this study is:

Guidelines for Determining Design Basis Ground Motions, Report TR-102293, Electric Power Research Institute, Palo Alto, California, November 1993, Sections 7 and 8.

The Robertson ELGX and 3ACS probes were used to collect long and short normal resistivity (LON, SHN), single point resistance (SPR) Spontaneous Potential (SP), natural gamma (NGAM) and caliper (CALP) data at 0.05-foot intervals to assist in delineating the transitions between different geologic units at the site.

INSTRUMENTATION

Suspension Instrumentation

Suspension soil velocity measurements were performed in each boring using the suspension PS logging system, manufactured by OYO Corporation, and their subsidiary, Robertson Geologging. This system directly determines the average velocity of a 3.3 feet high segment of the soil column surrounding the boring of interest by measuring the elapsed time between arrivals of a wave propagating upward through the soil column. The receivers that detect the wave, and the source that generates the wave, are moved as a unit in the boring producing relatively constant amplitude signals at all depths.

Winch GEOVision 7-conductor
Sheave - Measuring wheel GEOVision S/N 102
OYO 170 PS Logging unit M/N 3331 S/N 160024
OYO PS Logger Borehole Probe, includes:
Isolation tube, 1m Model 3387B S/N 280068
Weight Model 3302W S/N 470151
OYO PS 170 Source Model 3304 S/N 21050
Receiver/Sensor S/N 30086
Driver Model 3386A S/N 490157

Table 2 Suspension PS Logging Equipment

The suspension system probe consists of a combined reversible polarity solenoid horizontal shear-wave source (S_H) and compressional-wave source (P), joined to two biaxial receivers by a flexible isolation cylinder, as shown in Figure 1. The separation of the two receivers is 3.3 feet, allowing average wave velocity in the region between the receivers to be determined by inversion of the wave travel time between the two receivers. The total length of the probe as used in these surveys is 21 feet, with the center point of the receiver pair 12.1 feet above the bottom end of the probe.

The probe receives control signals from, and sends the receiver signals to, instrumentation on the surface via an armored 4- or 7-conductor cable. The cable is wound onto the drum of a winch and is used to support the probe. Cable travel is measured to provide probe depth data, using a 3.3-foot circumference sheave fitted with a digital rotary encoder.

The entire probe is suspended in the boring by the cable, therefore, source motion is not coupled directly to the boring walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling the boring and surrounding the source. This pressure wave is converted to P and S_H-waves in the surrounding soil and rock as it passes through the casing and grout annulus and impinges upon the wall of the boring. These waves propagate through the soil and rock surrounding the boring, in turn causing a pressure wave to be generated in the fluid surrounding the receivers as the soil waves pass their location. Separation of the P and S_H-waves at the receivers is performed using the following steps:

1. Orientation of the horizontal receivers is maintained parallel to the axis of the source, maximizing the amplitude of the recorded S_H -wave signals.
2. At each depth, S_H-wave signals are recorded with the source actuated in opposite directions, producing S_H-wave signals of opposite polarity, providing a characteristic S_H-wave signature distinct from the P-wave signal.
3. The 7.1-foot separation of source and receiver 1 permits the P-wave signal to pass and damp significantly before the slower S_H-wave signal arrives at the receiver. In faster soils or rock, the isolation cylinder is extended to allow greater separation of the P- and S_H-wave signals.
4. In saturated soils, the received P-wave signal is typically of much higher frequency than the received S_H-wave signal, permitting additional separation of the two signals by low pass filtering.
5. Direct arrival of the original pressure pulse in the fluid is not detected at the receivers because the wavelength of the pressure pulse in fluid is significantly greater than the dimension of the fluid annulus surrounding the probe, preventing significant energy transmission through the fluid medium.

In operation, a distinct, repeatable pattern of impulses is generated at each depth as follows:

1. The source is fired in one direction producing dominantly horizontal shear with some vertical compression, and the signals from the horizontal receivers situated parallel to the axis of motion of the source are recorded.
2. The source is fired again in the opposite direction and the horizontal receiver signals are recorded.
3. The source is fired again and the vertical receiver signals are recorded. The repeated source pattern facilitates the picking of the P and S_H-wave arrivals; reversal of the source changes the polarity of the S_H-wave pattern but not the P-wave pattern.

The data from each receiver during each source activation is recorded as a different channel on the recording system. The suspension PS system has six channels (two simultaneous recording channels), each with a 1024 sample record. The recorded data are displayed as six channels with a common time scale. Data are stored on disk for further processing. Up to 8 sampling sequences can be summed to improve the signal to noise ratio of the signals.

Review of the displayed data on the recorder or computer screen allows the operator to set the gains, filters, delay time, pulse length (energy), sample rate, and summing number to optimize the quality of the data before recording. Verification of the calibration of the suspension PS digital recorder is performed every twelve months using a NIST traceable frequency source and counter, as outlined in Appendix C.

Resistivity / Spontaneous Potential Instrumentation

Resistivity, spontaneous potential and natural gamma data were collected using a Model ELXG electric log probe, S/N 5490, manufactured by Robertson Geologging, Ltd. This probe measures Single Point Resistance (SPR), short normal (16-inch) resistivity, long normal (64-inch) resistivity and spontaneous potential (SP). The probe is 8.2 feet long, and 1.7 inches in diameter.

This probe is useful in the following studies:

- Bed boundary identification
- Strata correlation between borings
- Strata geometry and type (shale indication)

The probe receives control signals from, and sends the digitized measurement values to, a Robertson Micrologger II, on the surface via an armored 4 conductor cable. The cable is wound onto the drum of a winch and is used to support the probe. Cable travel is measured to provide probe depth data, using a 3.3-foot circumference sheave fitted with a digital rotary encoder. The probe and depth data are transmitted by USB link from the Micrologger unit to a laptop computer where it is displayed and stored on hard disk.

The resistivity section of the probe operates by driving a low frequency alternating current into the formation from the central SPR/DRIVE electrode. The current returns via the logging cable armor. To ensure adequate penetration of the formation the logging cable is insulated for approximately 33 feet from the cablehead. Voltages are measured between the 16-inch and 64-inch electrodes and the cable armor, as noted below:

- 16-inch normal (Short Normal-SHN): The survey current leaves the SPR/DRIVE electrode in all directions, diverging as it does so. In a homogeneous soil, an equipotential sphere with radius equal to the electrode spacing (16-inches) will define the volume of investigation. The voltage at the 16-inch electrode divided by the drive current produces a resistance value that is then multiplied by a geometric factor of approximately 12.5 times the electrode spacing, to produce a resistivity value in ohm-feet. The geometric factor is the quotient between the cross-sectional area the current passes through and the distance the current travels.

- 64 inch normal (Long Normal-LON): The survey current leaves the SPR/DRIVE electrode in all directions, diverging as it does so. In a homogeneous soil, an equipotential sphere with radius equal to the electrode spacing (64 inches) will define the volume of investigation. The voltage at the 64-inch electrode divided by the drive current produces a resistance value that is then multiplied by a geometric factor of approximately 12.5 times the electrode spacing, to produce a resistivity value in ohm-feet. The geometric factor is the quotient between the cross-sectional area the current passes through and the distance the current travels.

- Single Point Resistance (SPR): The current flowing to the cable armor is measured along with the voltage at the SPR/DRIVE electrode. The voltage divided by current gives resistance. SPR is a measurement of formation resistance only, not a volumetric measurement as Short and Long Normal are. SPR can usually resolve thinner beds, but is more readily influenced by boring diameter and changes in boring fluid.

- Spontaneous Potential (SP): This is the DC bias of the 16-inch electrode with respect to the voltage reference at the surface. Data quality is dependant upon good grounding at the surface, which is achieved with a copper clad steel stake driven into the mud-pit

Caliper / Natural Gamma Instrumentation

Caliper and natural gamma data were collected using a Model 3ACS 3-leg caliper probe, serial number 5368, manufactured by Robertson Geologging, Ltd. With the short arm configuration used in these surveys, the probe permitted measurement of boring diameters between 1.6 and 16 inches. With this tool, caliper measurements were collected concurrent with measurement of natural gamma emission from the boring walls. The probe is 6.82 feet long, and 1.5 inches in diameter.

This probe is useful in the following studies:

- Measurement of boring diameter and volume
- Location of hard and soft formations
- Location of fissures, caving, pinching and casing damage
- Bed boundary identification
- Strata correlation between borings

The probe receives control signals from, and sends the digitized measurement values to, a Robertson Micrologger II on the surface via an armored 4 conductor cable. The cable is wound onto the drum of a winch and is used to support the probe. Cable travel is measured to provide probe depth data, using a 3.28 foot circumference sheave fitted with a digital rotary encoder. The probe and depth data are transmitted by USB link from the Micrologger unit to a laptop computer where it is displayed and stored on hard disk.

The caliper consists of three arms, each with a toothed quadrant at their base, pivoted in the lower probe body. A toothed rack engages with each quadrant, thus constraining the arms to move together. Linear movement of the rack is converted to opening and closing of the arms. Springs hold the arms open in the operating position. A motor drive is provided to retract the arms, allowing the probe to be lowered into the boring. The rack is coupled to a potentiometer which converts movement into a voltage sensed by the probe's microprocessor.

Natural gamma measurements rely upon small quantities of radioactive material contained in all rocks to emit gamma radiation as they decay. Trace amounts of uranium and thorium are present in a few minerals, where potassium-bearing minerals such as feldspar, mica and clays will include traces of a radioactive isotope of potassium. These emit gamma radiation as they decay with an extremely long half-life. This radiation is detected by scintillation - the production of a tiny flash of light when gamma rays strike a crystal of sodium iodide. The light is converted into an electrical pulse by a photomultiplier tube. Pulses above a threshold value of 60 KeV are counted by the probe's microprocessor. The measurement is useful because the radioactive elements are concentrated in certain rock types e.g. clay or shales, and depleted in others e.g. sandstone or coal

MEASUREMENT PROCEDURES

Suspension Measurement Procedures

Each boring was logged uncased, filled with bentonite/polymer based drilling mud. Measurements followed the **GEO***Vision* Procedure for P-S Suspension Seismic Velocity Logging, revision 1.4 The probe was positioned with the top of the probe at the top of the mud box, and the electronic depth counter was set to 8.2 feet, the distance between the mid-point of the receiver and the top of the probe, minus the height of the mud box, as verified with a tape measure, and recorded on the field logs. The probe was then lowered to the bottom of the boring, stopping at 1.6-foot intervals to collect data, as summarized in Table 3.

At each measurement depth the measurement sequence of two opposite horizontal records and one vertical record was performed, and the gains were adjusted as required. The data from each depth were viewed on the computer display, checked, and recorded on disk before moving to the next depth.

Upon completion of the measurements, the probe zero depth indication at the depth reference point was verified prior to removal from the boring.

Resistivity / Spontaneous Potential Measurement Procedures

Each boring was logged uncased, filled with bentonite/polymer based drilling mud. The probe was connected to the logging cable using a 33-foot long insulating cable section or "yoke". The probe head was insulated by wrapping all exposed metal of the cablehead and probe with self-amalgamating insulation tape. The insulating yoke was checked for any damage, and repaired with self-amalgamating insulation tape as needed.

The reference ground stake was driven firmly into the mud pit, and connected to the ground socket on the winch switch box.

This sonde was not calibrated in the field, as it is used to provide qualitative measurements, not quantitative values, and is used only to assist in picking transitions between stratigraphic units, as described in ASTM D5753, "Planning and Conducting Borehole Geophysical Surveys". Prior to each logging run, the resistivity and SP functions were verified, using the manufacturer's supplied ELOG test fixture, which connects fixed resistance values across the probe electrodes to mimic soil resistivities of 10, 100, 1000 and 10000 ohm-feet, and an SP of 100 millivolts. The measured dimensions, as displayed on the recording computer screen, were recorded on the field log sheet, as well as a digital record, and compared with the test box values.

In each boring, the probe was positioned with the top of the probe at the top of the mud box, and the electronic depth counter was set to 8.2 feet, the specified length of the probe, minus the height of the mud box, as verified with a tape measure. The probe was lowered to the bottom of the boring, where data collection was begun. The probe was then returned to the surface at 10 feet/sec, collecting data continuously at 0.05-foot spacing, as summarized in Table 3.

Upon completion of the measurements, the probe zero depth indication at the depth reference point was verified prior to removal from the boring.

Caliper / Natural Gamma Measurement Procedures

Each boring was logged uncased, filled with bentonite/polymer based drilling mud. The probe was positioned with the top of the probe at the top of the mud box, and the electronic depth counter was set to 4.7 feet, the distance between the measuring point and the top of the probe, minus the height of the mud box, as verified with a tape measure, and recorded on the field logs. Measurements followed ASTM D6167-97 (Re-approved 2004) Conducting Borehole Geophysical Logging – Mechanical Caliper.

Prior to and following each logging run, the caliper tool was verified, using the manufacturer's supplied three point calibration jig. The three point jig is a circular plate with a series of holes in the top surface into which the tips of the caliper arms fit. This has circles of diameters from 2 to 12 inches. The calibration jig is placed over a bucket with the probe standing upright with its nose section passing through the jig's central hole. The caliper probe arms are opened under program control, and a log is recorded as the tips of the arms are placed in the holes on the calibration jig and inside the PVC coupling. The measured dimensions, as displayed on the recording computer screen was recorded on the field log sheet, as well as in the digital files, and compared with the calibration jig dimensions. If the verification records did not fall within +/- 0.05 inches of the calibration jig values, the caliper tool was re-calibrated, using the three point calibration jig, and the log repeated. As with the verification, the tips of the caliper arms are placed in the holes marked with the required diameter. During calibration, the value of the current calibration point, as stamped on the jig, is entered via the control computer. The system counts for 15 seconds to make an average of the response. The procedure is repeated for the second and third required openings.

The computation and generation of the calibration coefficient file is entirely automatic. The calibration file is the set of coefficients of a quadratic curve which fits the three data points.

Natural gamma was not calibrated in the field, as it is a qualitative measurement, not a quantitative value, and is used only to assist in picking transitions between stratigraphic units, as described in ASTM D6274-98 (Re-approved 2004), Conducting Borehole Geophysical Logging - Gamma.

In each boring, the probe was positioned with the top of the probe at the top of the casing, and the electronic depth counter was set to the specified length of the probe, minus the height of the casing stick-up, as verified with a tape measure, and recorded on the field logs. The probe was lowered to the bottom of the boring, where the caliper legs were opened, and data collection begun. The probe was then returned to the surface at 10 feet/minute, collecting data continuously at 0.05 foot spacing, as summarized in Table 3.

Upon completion of the measurements, the probe zero depth indication at the depth reference point was verified prior to removal from the boring.

BORING NUMBER	TOOL AND RUN NUMBER	DEPTH RANGE (FEET)	OPEN HOLE (FEET)	DEPTH TO BOTTOM OF CASING (FEET)	SAMPLE INTERVAL (FEET)	DATE LOGGED
BH-1-10	SUSPENSION 1	27.9 – 182.1	-	25	1.6	3/31/2010
BH-1-10	ELOG 1	198.9 – 39.6	196.9	25	0.05	3/31/2010
BH-1-10	CALIPER/GAMMA 1	182.8 – 12.6	-	25	0.05	3/31/2010
BH-2-10	SUSPENSION 1	23.0 – 180.4	-	20	1.6	4/2/2010
BH-2-10	ELOG 1	193.9 – 39.3	193.9	20	0.05	4/2/2010
BH-2-10	CALIPER/GAMMA 1	182.4 – 10.5	-	20	0.05	4/2/2010

- PROBE DID NOT TOUCH BOTTOM OF BORING

Table 3. Logging dates and depth ranges

DATA ANALYSIS

Suspension Analysis

Using the proprietary OYO program PSLOG.EXE version 1.0, the recorded digital waveforms were analyzed to locate the most prominent first minima, first maxima, or first break on the vertical axis records, indicating the arrival of P-wave energy. The difference in travel time between receiver 1 and receiver 2 (R1-R2) arrivals was used to calculate the P-wave velocity for that 3.3-foot segment of the soil column. When observable, P-wave arrivals on the horizontal axis records were used to verify the velocities determined from the vertical axis data. The time picks were then transferred into an EXCEL template (EXCEL version 2003 SP2) to complete the velocity calculations based upon the arrival time picks made in PSLOG.

The P-wave velocity over the 7.1-foot interval from source to receiver 1 (S-R1) was also picked using PSLOG, and calculated and plotted in EXCEL, for quality assurance of the velocity derived from the travel time between receivers. In this analysis, the depth values as recorded were increased by 5.1 feet to correspond to the mid-point of the 7.1-foot S-R1 interval. Travel times were obtained by picking the first break of the P-wave signal at receiver 1 and subtracting 0.3 milliseconds, the calculated and experimentally verified delay from source trigger pulse (beginning of record) to source impact. This delay corresponds to the duration of acceleration of the solenoid before impact.

As with the P-wave records, using PSLOG, the recorded digital waveforms were analyzed to locate the presence of clear S_H-wave pulses, as indicated by the presence of opposite polarity pulses on each pair of horizontal records. Ideally, the S_H-wave signals from the 'normal' and 'reverse' source pulses are very nearly inverted images of each other. Digital FFT - IFFT lowpass filtering was used to remove the higher frequency P-wave signal from the S_H-wave signal. Different filter cutoffs were used to separate P- and S_H-waves at different depths, ranging from 600 Hz in the slowest zones to 2000 Hz in the regions of highest velocity. At each depth, the filter frequency was selected to be at least twice the fundamental frequency of the S_H-wave signal being filtered.

Generally, the first maxima were picked for the 'normal' signals and the first minima for the 'reverse' signals, although other points on the waveform were used if the first pulse was distorted. The absolute arrival time of the 'normal' and 'reverse' signals may vary by +/- 0.2 milliseconds, due to differences in the actuation time of the solenoid source caused by constant mechanical bias in the source or by boring inclination. This variation does not affect the R1-R2 velocity determinations, as the differential time is measured between arrivals of waves created by the same source actuation. The final velocity value is the average of the values obtained from the 'normal' and 'reverse' source actuations.

As with the P-wave data, S_H-wave velocity calculated from the travel time over the 7.1-foot interval from source to receiver 1 was calculated and plotted for verification of the velocity derived from the travel time between receivers. In this analysis, the depth values were increased by 5.1 feet to correspond to the mid-point of the 7.1-foot S-R1 interval. Travel times were obtained by picking the first break of the S_H-wave signal at the near receiver and subtracting 0.3 milliseconds, the calculated and experimentally verified delay from the beginning of the record at the source trigger pulse to source impact.

These data and analysis were reviewed by John Diehl as a component of **GEO***Vision*'s in-house QA-QC program.

Figure 3 shows an example of R1 - R2 measurements on a sample filtered suspension record. In Figure 3, the time difference over the 3.3-foot interval of 1.88 milliseconds for the horizontal signals is equivalent to an S_H-wave velocity of 1745 feet/second. Whenever possible, time differences were determined from several phase points on the S_H-waveform records to verify the data obtained from the first arrival of the S_H-wave pulse. Figure 4 displays the same record before filtering of the S_H-waveform record with a 1400 Hz FFT - IFFT digital lowpass filter, illustrating the presence of higher frequency P-wave energy at the beginning of the record, and distortion of the lower frequency S_H-wave by residual P-wave signal.

Resistivity / Spontaneous Potential Analysis

No analysis is required with the resistivity or spontaneous potential data. Using Robertson Geologging Winlogger software version 1.5, build 401J, these data were converted to LAS 2.0 and PDF formats for transmittal to the client.

Caliper / Natural Gamma Analysis

No analysis is required with the caliper or natural gamma data. Using Robertson Geologging Winlogger software version 1.5, build 401J, these data were converted to LAS 2.0 and PDF formats for transmittal to the client.

RESULTS

Suspension Results

Suspension R1-R2 P- and S_H-wave velocities are plotted in Figures 4 and 6. The suspension velocity data presented in these figures are presented in Tables 4 and 5. These plots and data are included in the EXCEL analysis files transmitted separately.

P- and S_H-wave velocity data from R1-R2 analysis and quality assurance analysis of S-R1 data are plotted together in Figures A-1 and A-2 to aid in visual comparison. It should be noted that R1-R2 data are an average velocity over a 3.3 feet segment of the soil column; S-R1 data are an average over 7.1 feet, creating a significant smoothing relative to the R1-R2 plots. S-R1 data are presented in Tables A-1 and A-2, and included in the EXCEL analysis files transmitted separately.

Calibration procedures and records for the suspension PS measurement system are presented in Appendix C.

Resistivity / Spontaneous Potential Results

Resistivity and spontaneous potential data is presented as single page logs in Figures 5 and 7, as well as multi-page logs in Appendix B, and delivered as LAS 2.0 data and Acrobat files transmitted separately.

Caliper / Natural Gamma Results

Caliper and natural gamma data for borings are presented as single page logs in Figures 5 and 7, as well as a multi-page logs in Appendix B, and delivered as LAS 2.0 data and Acrobat files transmitted separately.

SUMMARY

Discussion of Suspension Results

Suspension PS velocity data are ideally collected in an uncased fluid filled boring, drilled with rotary mud (rotary wash) methods. These borings were generally well suited for collection of suspension PS velocity data.

Suspension PS velocity data quality is judged based upon 5 criteria:

1. Consistent data between receiver to receiver (R1 – R2) and source to receiver (S – R1) data.
2. Consistent relationship between P-wave and S_H -wave (excluding transition to saturated soils)
3. Consistency between data from adjacent depth intervals.
4. Clarity of P-wave and S_H-wave onset, as well as damping of later oscillations.
5. Consistency of profile between adjacent borings, if available.

These data show good correlation between R1 – R2 and S – R1 data, as well as good correlation between P-wave and S_H-wave velocities for all borings. Adjacent depth intervals provide similar velocities, indicating fairly homogeneous materials in most depth intervals. P-wave and S_H-wave onsets are generally clear, and later oscillations are well damped.

Discussion of Resistivity / Spontaneous Potential Results

The resistivity logs show significant changes in lithology in all borings, corresponding with changes in velocity. SP logs are not particularly diagnostic at this site. The electrical data are not valid above 39 feet, as the upper yoke electrode moves above the drilling fluid and precludes the collection of electrical data.

Discussion of Caliper / Natural Gamma Results

The caliper logs show fairly even gauge in most of the borings, with a few washouts at shallow depths. Natural gamma logs do not show substantial changes in any of the borings, but there are several borings that show minor changes with lithology, and correspond to changes in velocity..

Quality Assurance

These boring geophysical measurements were performed using industry-standard or better methods for measurements and analyses. All work was performed under **GEO***Vision* quality assurance procedures, which include:

- Use of NIST-traceable calibrations, where applicable, for field and laboratory instrumentation
- Use of standard field data logs
- Use of independent verification of velocity data by comparison of receiver-to-receiver and source-to-receiver velocities
- Independent review of calculations and results by a registered professional engineer, geologist, or geophysicist.

Suspension Data Reliability

P- and S_H-wave velocity measurement using the Suspension Method gives average velocities over a 3.3 feet interval of depth. This high resolution results in the scatter of values shown in the graphs. Individual measurements are very reliable with estimated precision of +/- 5%. Standardized field procedures and quality assurance checks contribute to the reliability of these data.

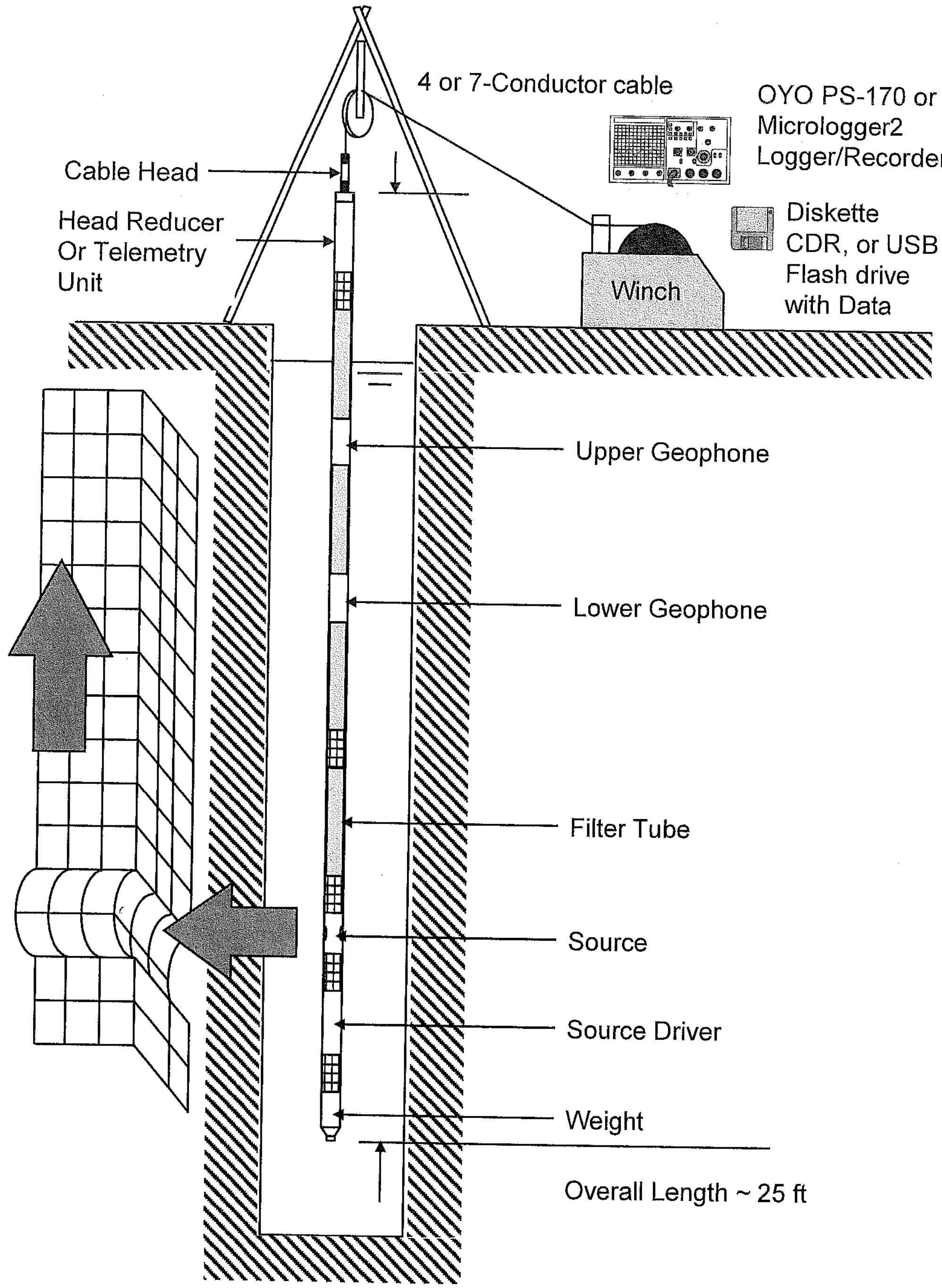


Figure 1: Concept illustration of P-S logging system

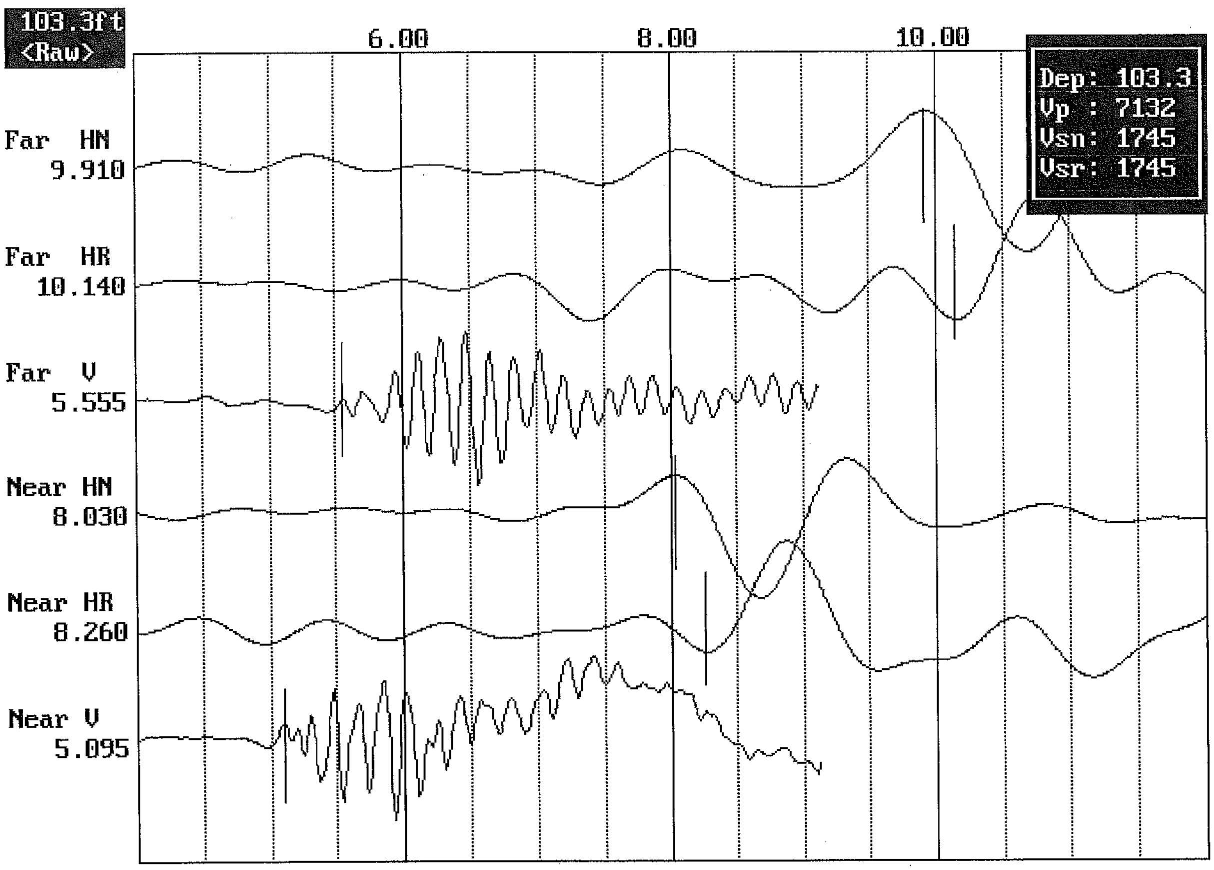


Figure 2: Example of filtered (1400 Hz lowpass) record

Figure 3. Example of unfiltered record

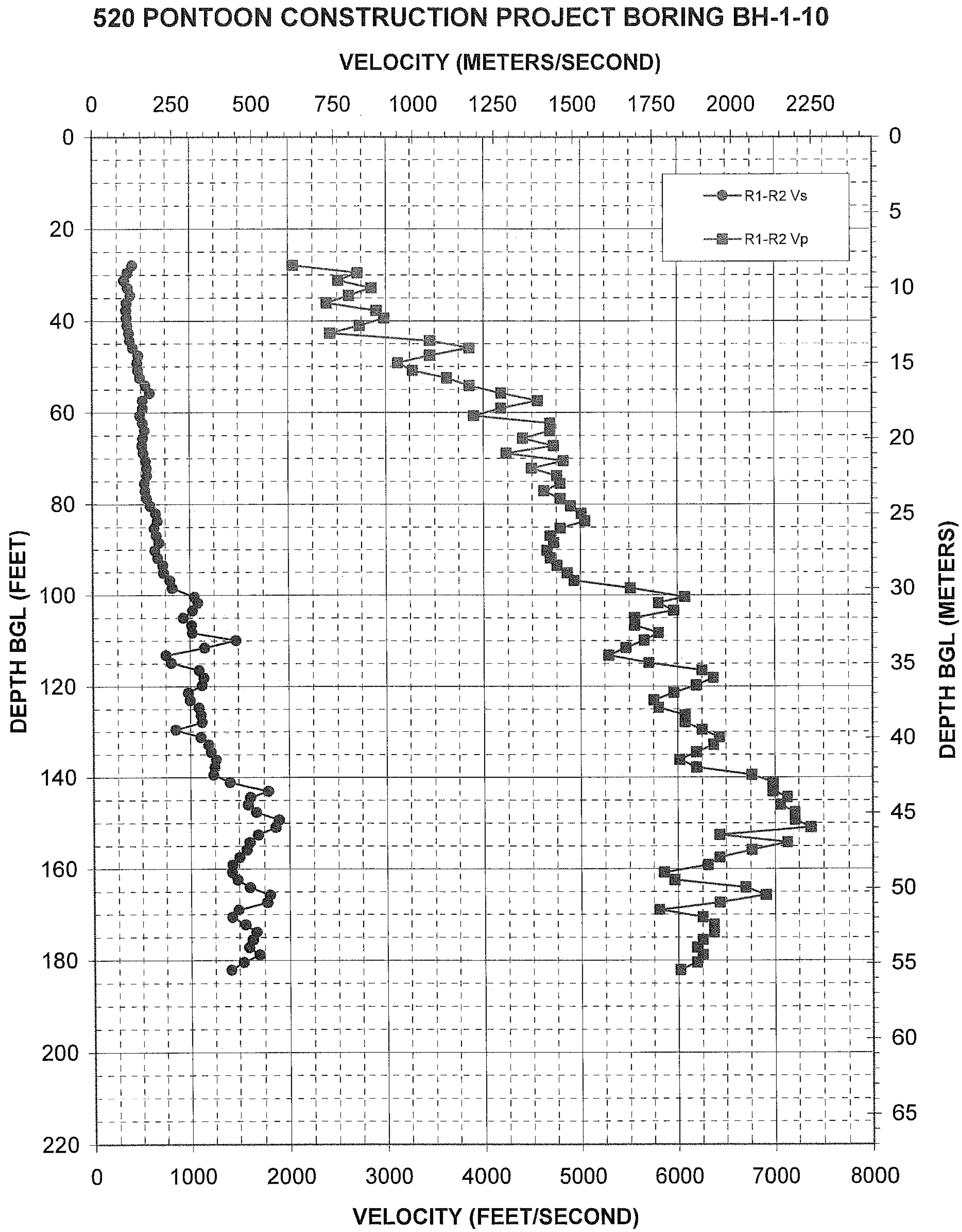


Figure 4: Boring BH-1-10, Suspension R1-R2 P- and S_H-wave velocities

Depth (feet)	V_s (feet/sec)	V_p (feet/sec)
27.9	411	2051
29.5	364	2711
31.2	325	2514
32.8	365	2853
34.4	393	2625
36.1	353	2395
37.7	348	2903
39.4	352	2983
41.0	361	2734
42.7	377	2430
44.3	386	3454
45.9	417	3860
47.6	474	3454
49.2	462	3125
50.9	470	3281
52.5	492	3625
54.1	547	3860
55.8	594	4179
57.4	519	4557
59.1	519	4179
60.7	492	3906
62.3	519	4687
64.0	538	4687
65.6	521	4404
67.3	513	4721
68.9	525	4233
70.5	549	4825
72.2	558	4494
73.8	563	4755
75.5	536	4790
77.1	542	4621
78.7	558	4790
80.4	597	4897
82.0	653	5009
83.7	670	5047
85.3	637	4790
86.9	659	4687
88.6	687	4721
90.2	643	4654
91.9	673	4687
93.5	725	4755
95.1	733	4861
96.8	796	4934
98.4	820	5514
100.4	1048	6076
101.7	1083	5807
103.3	1028	5965
105.0	932	5561
106.6	1019	5561
108.3	1025	5807

Depth (feet)	V_s (feet/sec)	V_p (feet/sec)
111.5	1151	5468
113.2	754	5292
114.8	808	5706
116.5	1094	6249
118.1	1143	6371
119.8	1124	6190
121.4	982	5965
123.0	1000	5756
124.7	1090	5807
126.3	1112	6076
128.0	1124	6076
129.6	852	6249
131.2	1108	6433
132.9	1189	6371
134.5	1215	6190
136.2	1267	6020
137.8	1252	6190
139.4	1238	6765
141.1	1408	6981
143.0	1793	6981
144.4	1616	7132
146.0	1593	7056
147.6	1674	7211
149.3	1896	7211
150.9	1864	7373
152.6	1691	6433
154.2	1608	7132
155.8	1577	6765
157.5	1505	6433
159.1	1433	6309
160.8	1426	5859
162.4	1485	5965
164.0	1608	6696
165.7	1803	6907
167.3	1778	6433
169.0	1491	5807
170.6	1426	6249
172.2	1562	6371
173.9	1670	6371
175.5	1632	6249
177.2	1597	6190
178.8	1700	6249
180.4	1540	6190
182.1	1414	6020

Table 4. Boring BH-1-10, Suspension R1-R2 depths and P- and S_H-wave velocities

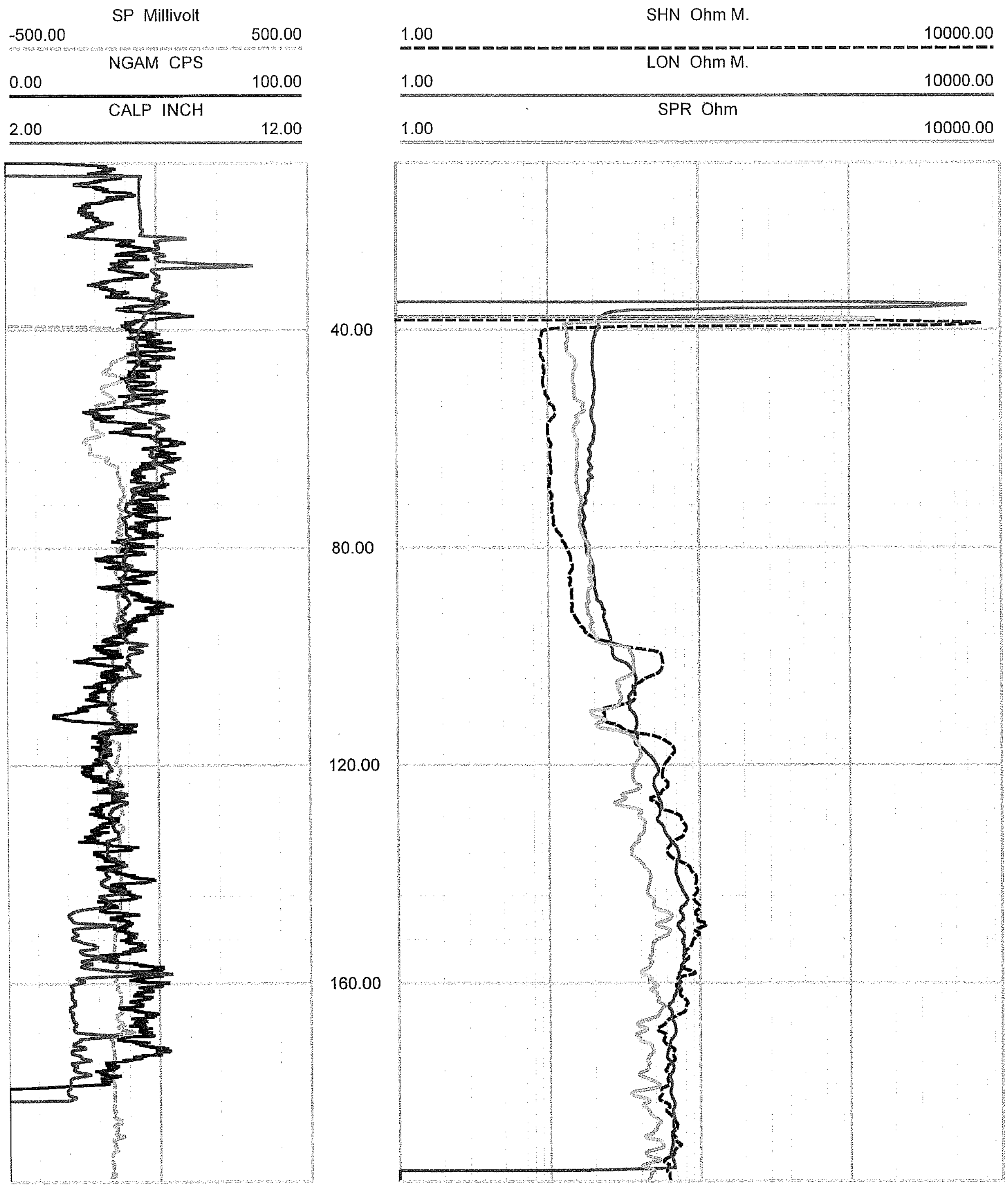


Figure 5. Boring BH-1-10, ELOG, Caliper and Natural Gamma logs

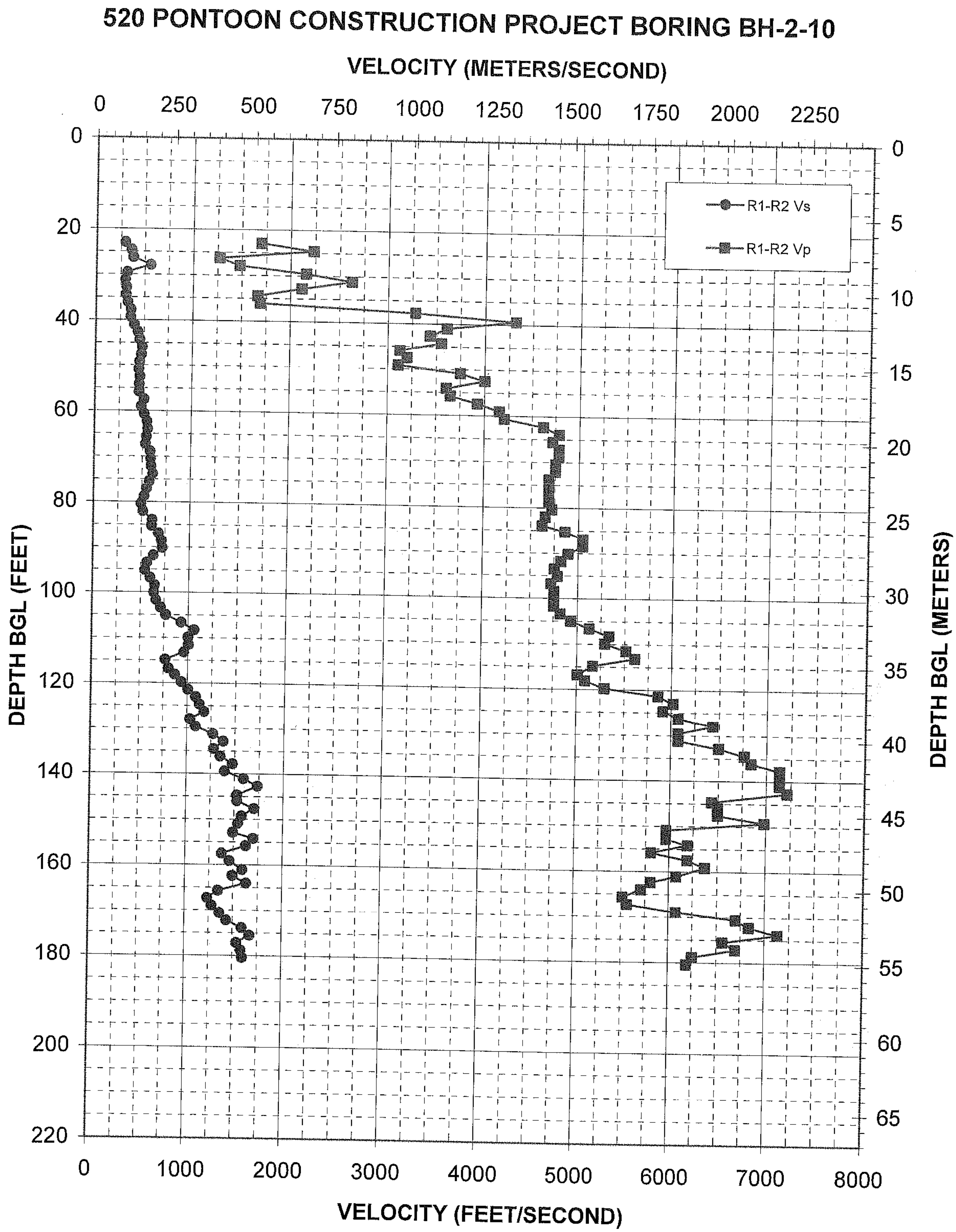


Figure 6: Boring BH-2-10, Suspension R1-R2 P- and S_H-wave velocities

Depth (feet)	V_s (feet/sec)	V_p (feet/sec)
23.0	294	1700
24.6	361	2232
26.2	379	1272
27.9	563	1478
29.5	319	2158
31.2	297	2625
32.8	310	2117
34.4	308	1665
36.1	327	1691
37.7	362	3281
39.4	360	4317
41.0	398	3605
42.7	437	3435
44.3	457	3547
45.9	484	3125
47.6	477	3201
49.2	457	3110
50.9	451	3750
52.5	464	4001
54.1	457	3605
55.8	457	3645
57.4	511	3929
59.1	482	4153
60.7	515	4206
62.3	545	4621
64.0	554	4790
65.6	540	4721
67.3	531	4790
68.9	581	4790
70.5	588	4755
72.2	594	4755
73.8	613	4687
75.5	578	4687
77.1	549	4687
78.7	525	4687
80.4	493	4721
82.0	515	4654
84.0	613	4621
85.3	608	4861
86.9	679	5047
88.6	713	5047
90.2	723	4897
91.9	630	4825
93.5	563	4755
95.1	542	4790
96.8	599	4721
98.4	650	4755
100.1	640	4755
101.7	663	4755
103.3	713	4825
105.0	768	4934
106.6	927	5126
108.3	1062	5335
109.9	1000	5292
111.5	1006	5514
113.2	959	5608
114.8	768	5167
116.8	802	5009
118.1	868	5087
119.8	937	5292
121.4	1006	5859
123.0	1086	6020
124.7	1127	5911
126.3	1176	6076
128.0	1032	6433
129.6	1090	6076
131.2	1272	6076
132.9	1379	6497
134.5	1282	6765
136.2	1350	6835
137.8	1478	7132
139.4	1396	7132
141.1	1593	7132
142.7	1736	7211
144.7	1526	6433
146.0	1530	6497
147.6	1704	6497
149.3	1577	6981
150.9	1540	5965
152.9	1491	5965
154.2	1700	6190
155.8	1624	5807
157.5	1379	6190
159.1	1458	6371
161.1	1593	6076
162.4	1491	5807
164.0	1632	5706
165.7	1345	5514
167.3	1233	5561
169.0	1282	6076
170.6	1361	6696
172.2	1433	6835
173.9	1593	7132
175.5	1674	6562
177.2	1540	6696
178.8	1585	6249
180.4	1600	6190

Table 5. Boring BH-2-10, Suspension R1-R2 depths and P- and S_H-wave velocities

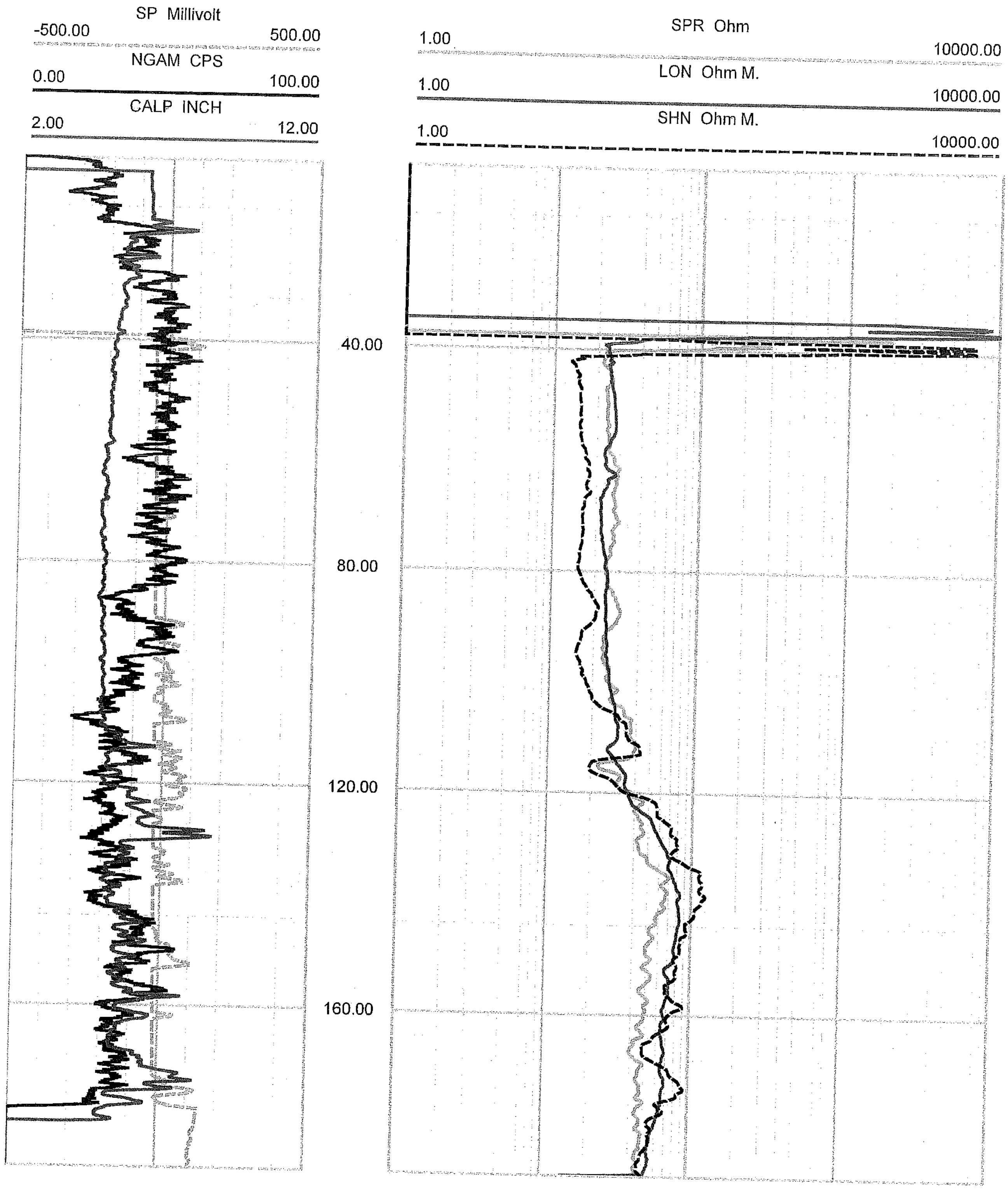


Figure 7. Boring BH-2-10, ELOG, Caliper and Natural Gamma logs

APPENDIX A

SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS

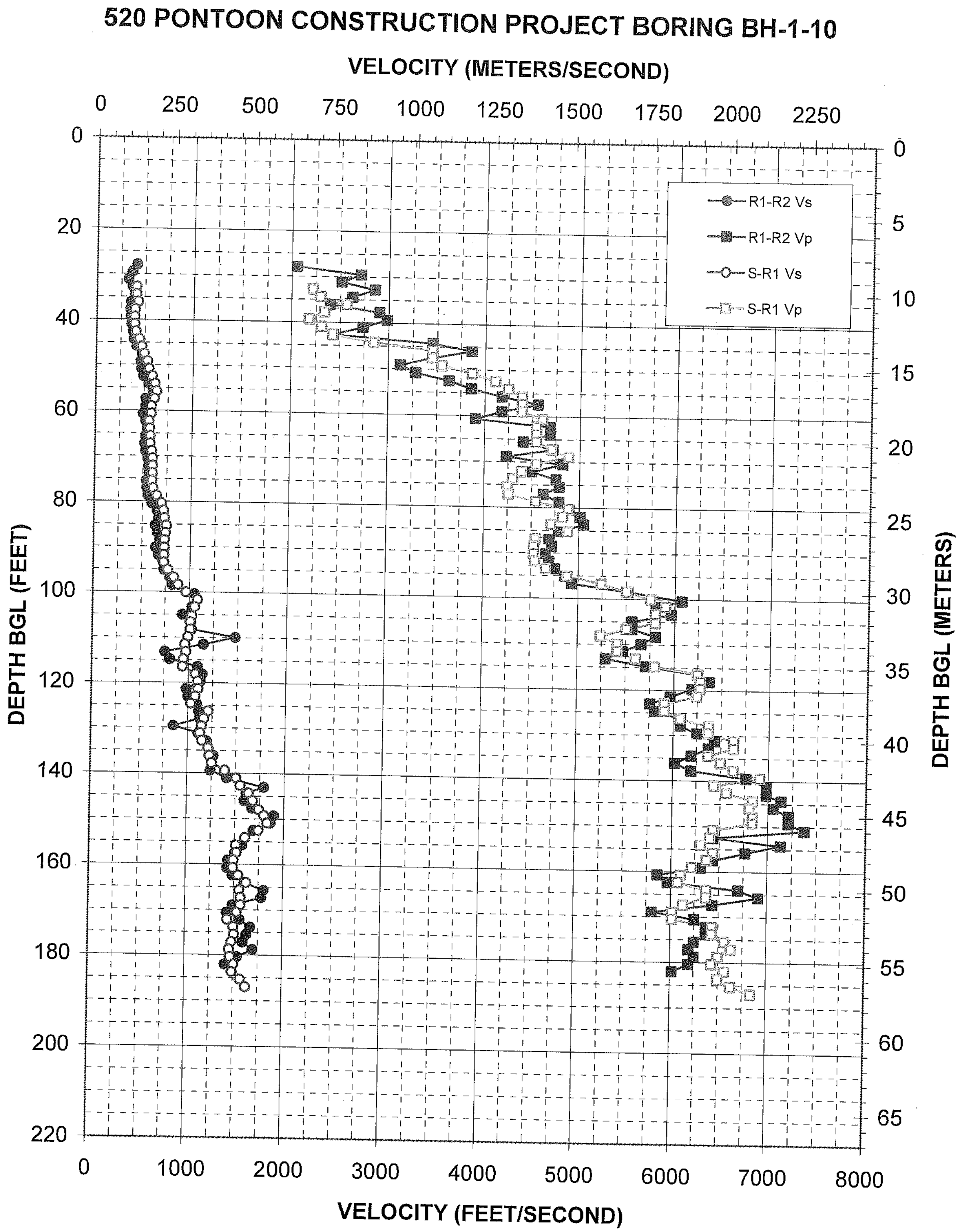


Figure A-1. Boring BH-1-10, R1 - R2 high resolution analysis and S - R1 quality assurance analysis P- and S_H-wave data

Depth (feet)	V_s (feet/sec)	V_p (feet/sec)
32.8	408	2217
34.4	409	2295
36.1	428	2568
37.7	395	2337
39.3	391	2180
41.0	392	2308
42.6	407	2424
44.3	443	2849
45.9	469	3455
47.5	497	3455
49.2	531	3550
50.8	549	3867
52.5	584	4111
54.1	614	4246
55.7	632	4389
57.4	608	4389
59.0	578	4389
60.7	578	4607
62.3	558	4543
63.9	563	4543
65.6	571	4543
67.2	571	4707
68.9	589	4884
70.5	597	4543
72.1	600	4389
73.8	600	4302
75.4	594	4246
77.1	608	4260
78.7	648	4543
80.3	700	4884
82.0	721	4830
83.6	732	4707
85.3	753	4884
86.9	744	4543
88.5	736	4527
90.2	736	4527
91.8	732	4543
93.5	732	4657
95.1	775	4884
96.8	835	5239
98.4	880	5505
100.0	965	5749
101.7	1083	5906
103.3	1055	5800
105.3	1027	5800
106.6	1012	5505
108.2	1012	5239
109.9	990	5413
111.5	961	5413
113.2	972	5600
114.8	947	5800
116.4	939	6246
118.1	1069	6276
119.7	1097	6276
121.4	1101	6246
123.0	1073	5906
124.6	1031	5906
126.3	1207	6071
127.9	1164	6369
129.6	1144	6369
131.2	1128	6629
132.8	1148	6629
134.5	1212	6369
136.1	1225	6496
137.8	1249	6629
139.4	1388	6911
141.0	1507	6432
142.7	1550	6562
144.3	1636	6838
146.0	1679	6802
147.9	1742	6838
149.2	1799	6838
150.9	1830	6432
152.5	1742	6400
154.2	1608	6307
155.8	1514	6432
157.4	1521	6369
159.1	1493	6216
160.7	1487	6100
162.4	1543	6071
164.0	1624	6369
165.6	1554	6369
167.3	1569	6128
168.9	1569	6015
170.6	1532	6015
172.2	1437	6432
173.9	1500	6432
175.5	1507	6562
177.1	1483	6629
178.8	1460	6496
180.4	1470	6432
182.1	1507	6562
183.7	1490	6496
185.3	1573	6629
187.0	1628	6838

Table A-1. Boring BH-1-10, S - R1 quality assurance analysis P- and S_H-wave data

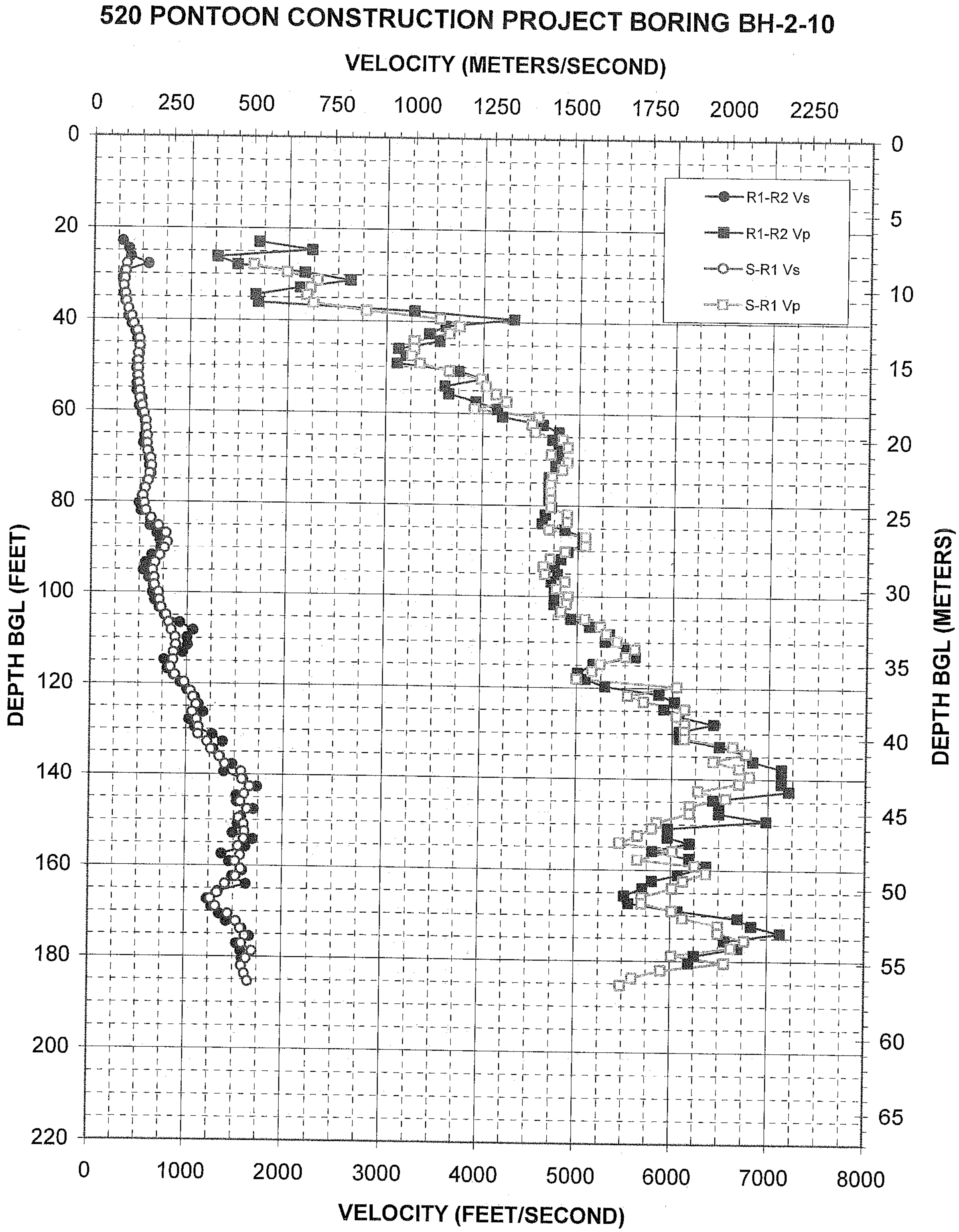


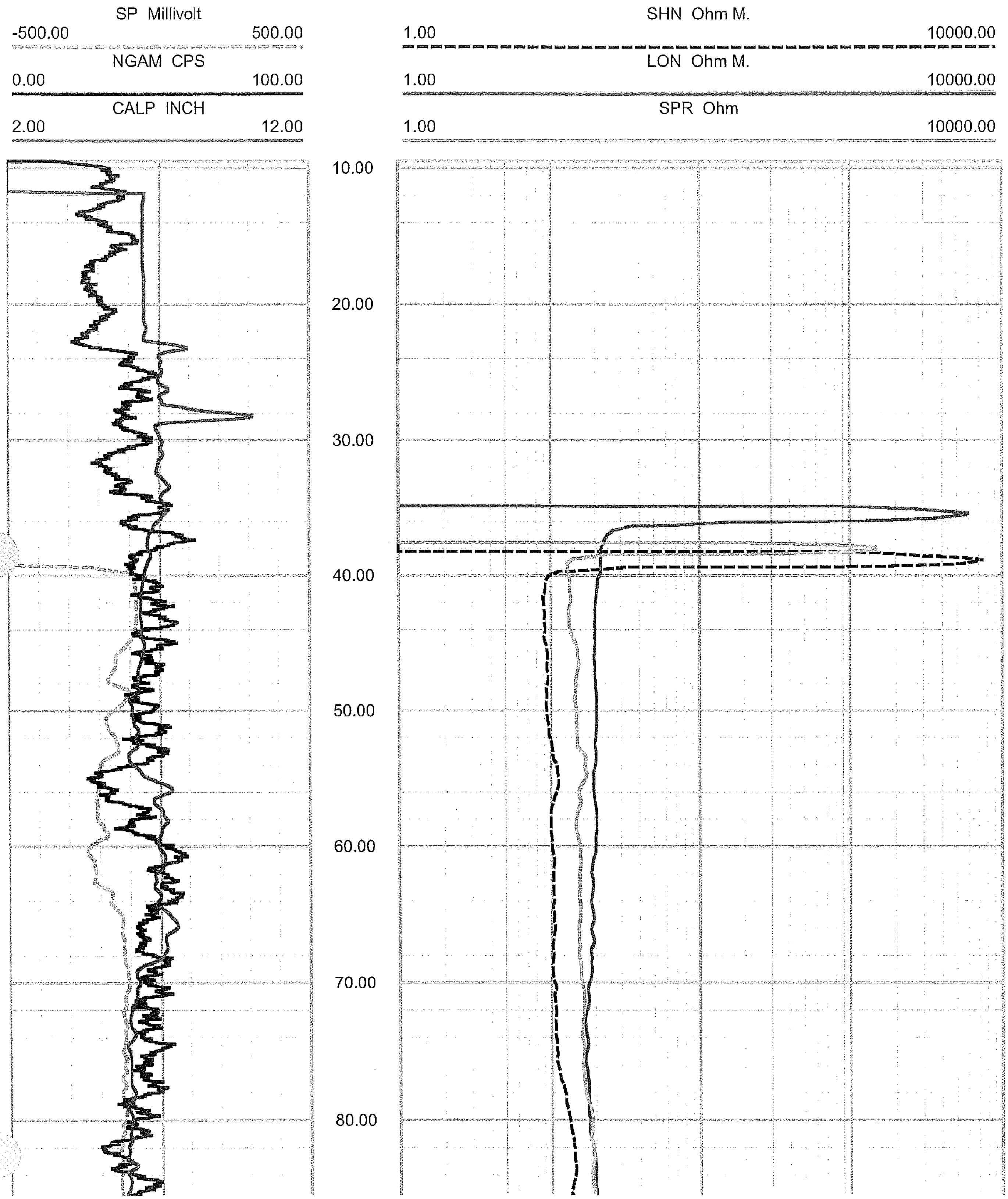
Figure A-2. Boring BH-2-10, R1 - R2 high resolution analysis and S - R1 quality assurance analysis P- and S_H-wave data

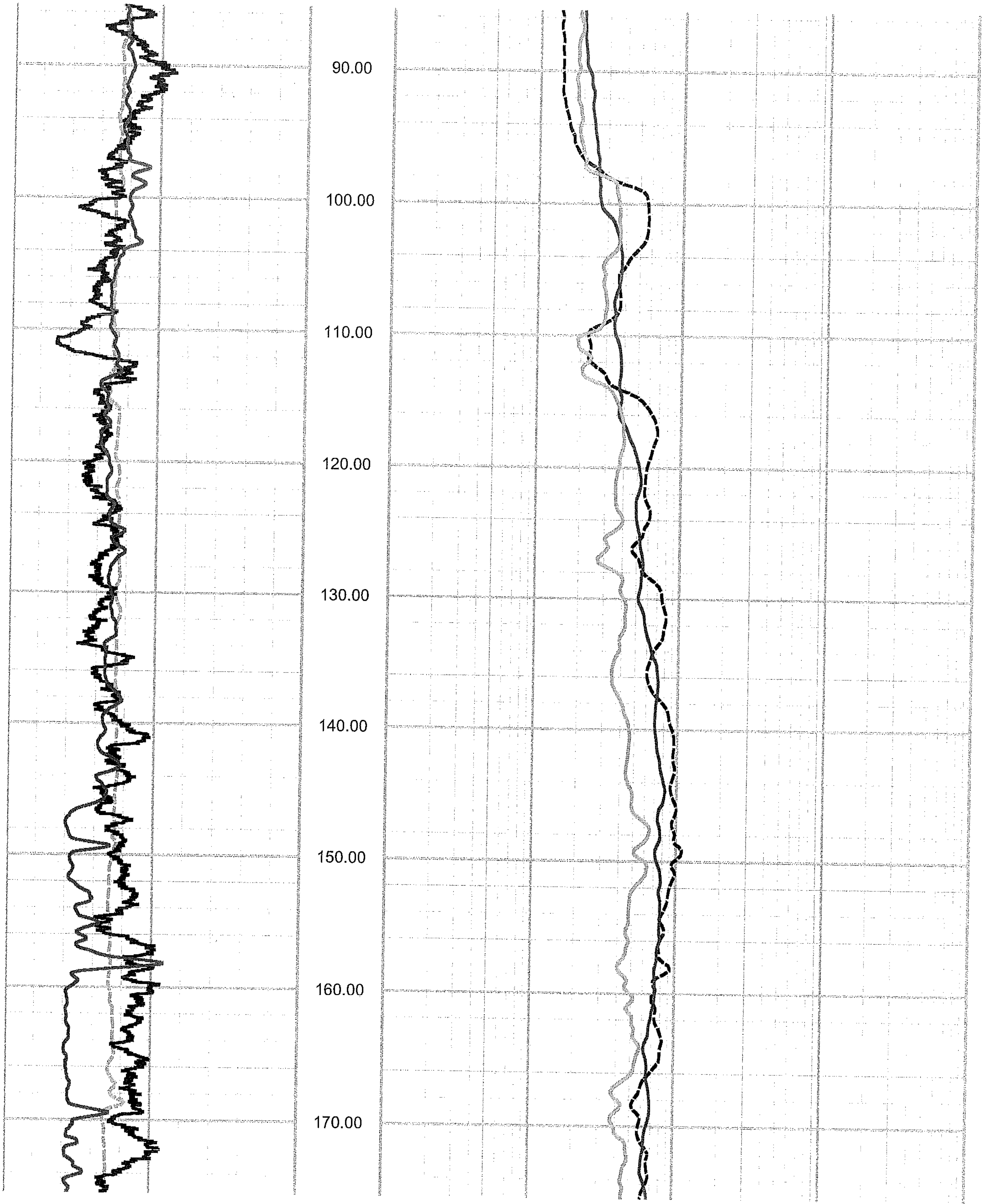
Depth (feet)	V_s (feet/sec)	V_p (feet/sec)
27.9	339	1645
29.5	330	1981
31.1	322	2287
32.8	308	2217
34.4	328	2180
36.1	339	2248
37.7	361	2788
39.3	397	3550
41.0	418	3755
42.6	458	3649
44.3	479	3281
45.9	480	3281
47.5	468	3256
49.2	468	3340
50.8	470	3649
52.5	471	3985
54.1	477	4035
55.7	487	4138
57.4	489	4246
59.0	512	3913
60.7	536	4575
62.3	556	4511
63.9	559	4543
65.6	571	4830
67.2	571	4884
68.9	573	4707
70.5	608	4884
72.1	617	4830
73.8	600	4724
75.4	586	4707
77.1	556	4707
78.7	531	4707
80.3	547	4707
82.0	563	4884
83.6	617	4884
85.3	696	4690
86.9	775	5075
88.9	789	5075
90.2	761	4866
91.8	714	4707
93.5	664	4640
95.1	639	4657
96.8	657	4866
98.4	660	4777
100.0	704	4903
101.7	712	4884
103.3	732	4830
105.0	785	5075
106.6	812	5239
108.2	839	5303
109.9	880	5413
111.5	887	5600
113.2	866	5505
114.8	862	5239
116.4	833	5156
118.1	880	4997
119.7	975	6043
121.7	1041	5529
123.0	1068	5698
124.6	1101	6128
126.3	1061	6043
127.9	1109	6128
129.6	1128	6128
131.2	1124	6128
132.8	1219	6629
134.5	1259	6767
136.1	1345	6432
137.8	1403	6697
139.4	1569	6802
141.0	1581	6697
142.7	1661	6276
144.3	1604	6562
146.0	1565	6187
147.6	1636	6187
149.6	1550	5852
150.9	1604	5800
152.5	1612	5649
154.2	1604	5459
155.8	1547	6015
157.8	1569	5649
159.1	1518	6246
160.7	1577	6369
162.4	1525	6128
164.0	1418	6015
166.0	1337	5698
167.3	1269	5698
168.9	1320	6015
170.6	1448	6128
172.2	1532	6496
173.9	1577	6496
175.5	1626	6767
177.1	1592	6629
178.8	1701	6015
180.4	1640	6562
182.1	1592	5906
183.7	1624	5600
185.3	1657	5482

Table A-2. Boring BH-2-10, S - R1 quality assurance analysis P- and S_H-wave data

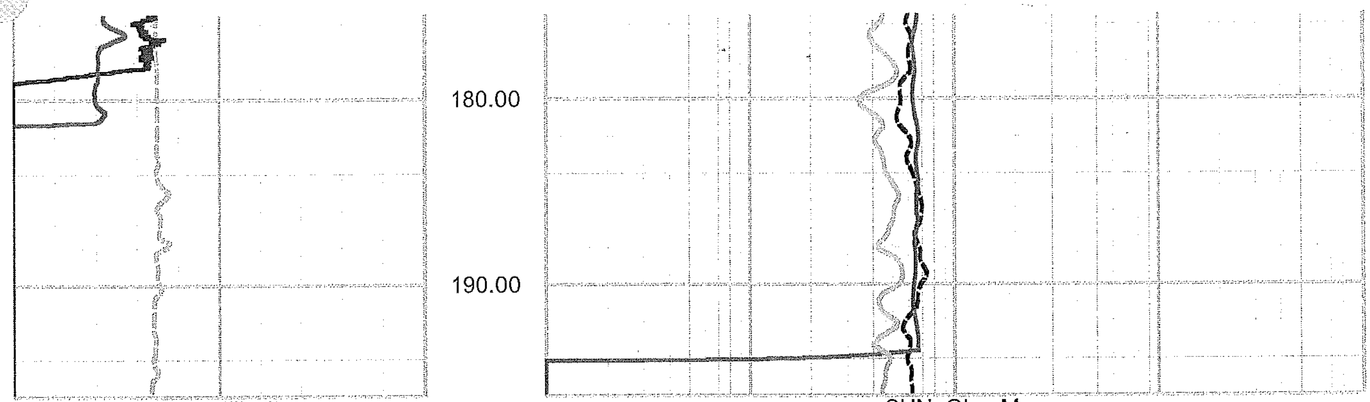
APPENDIX B

ELOG, CALIPER AND NATURAL GAMMA LOGS


SP Millivolt
-500.00
500.00
NGAM CPS
0.00
100.00
CALP INCH
2.00
12.00
SHN Ohm M.
1.00
10000.00
LON Ohm M.
1.00
10000.00
SPR Ohm
1.00
10000.00
10.00
20.00
30.00
40.00
50.00
60.00
70.00
80.00

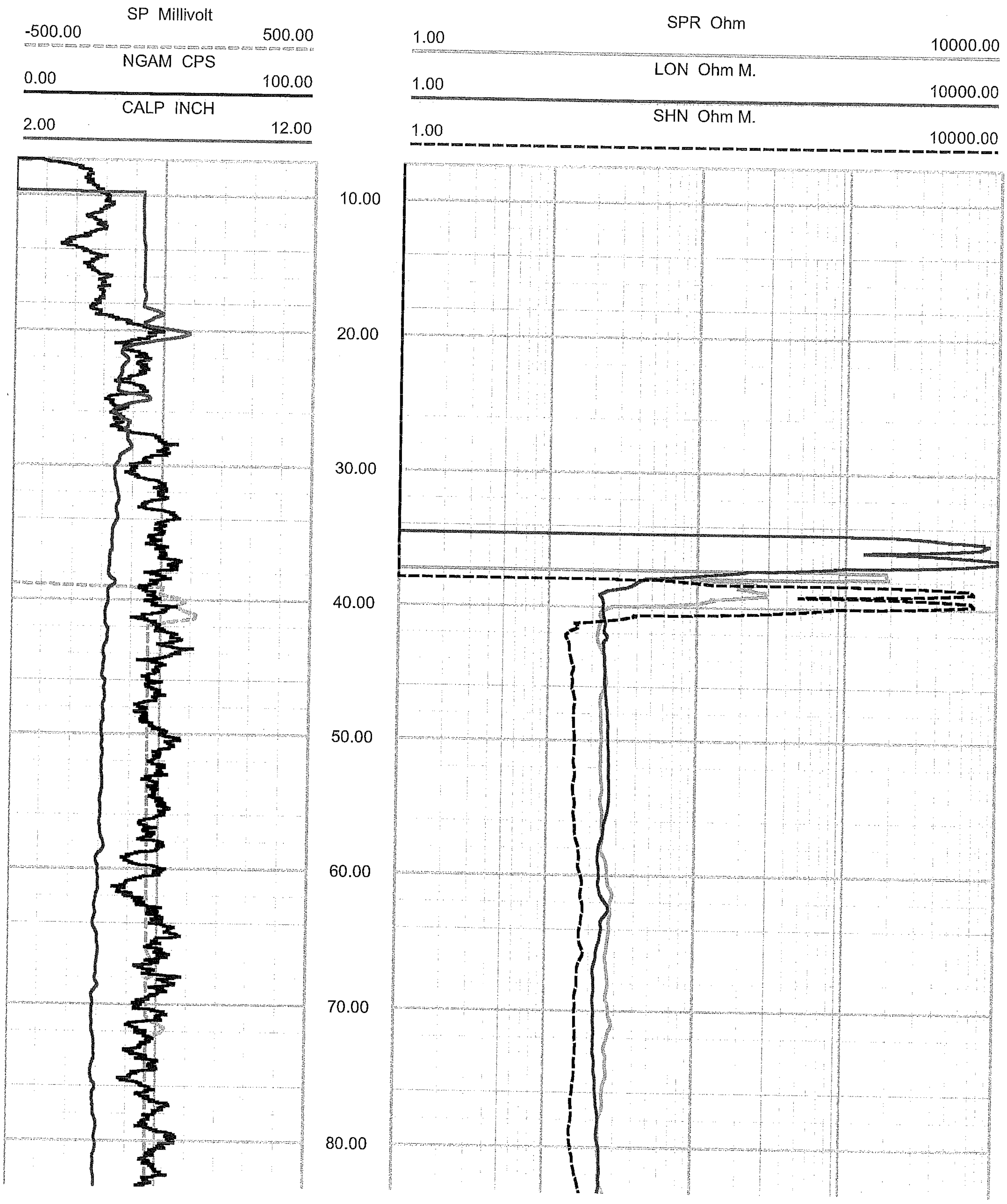


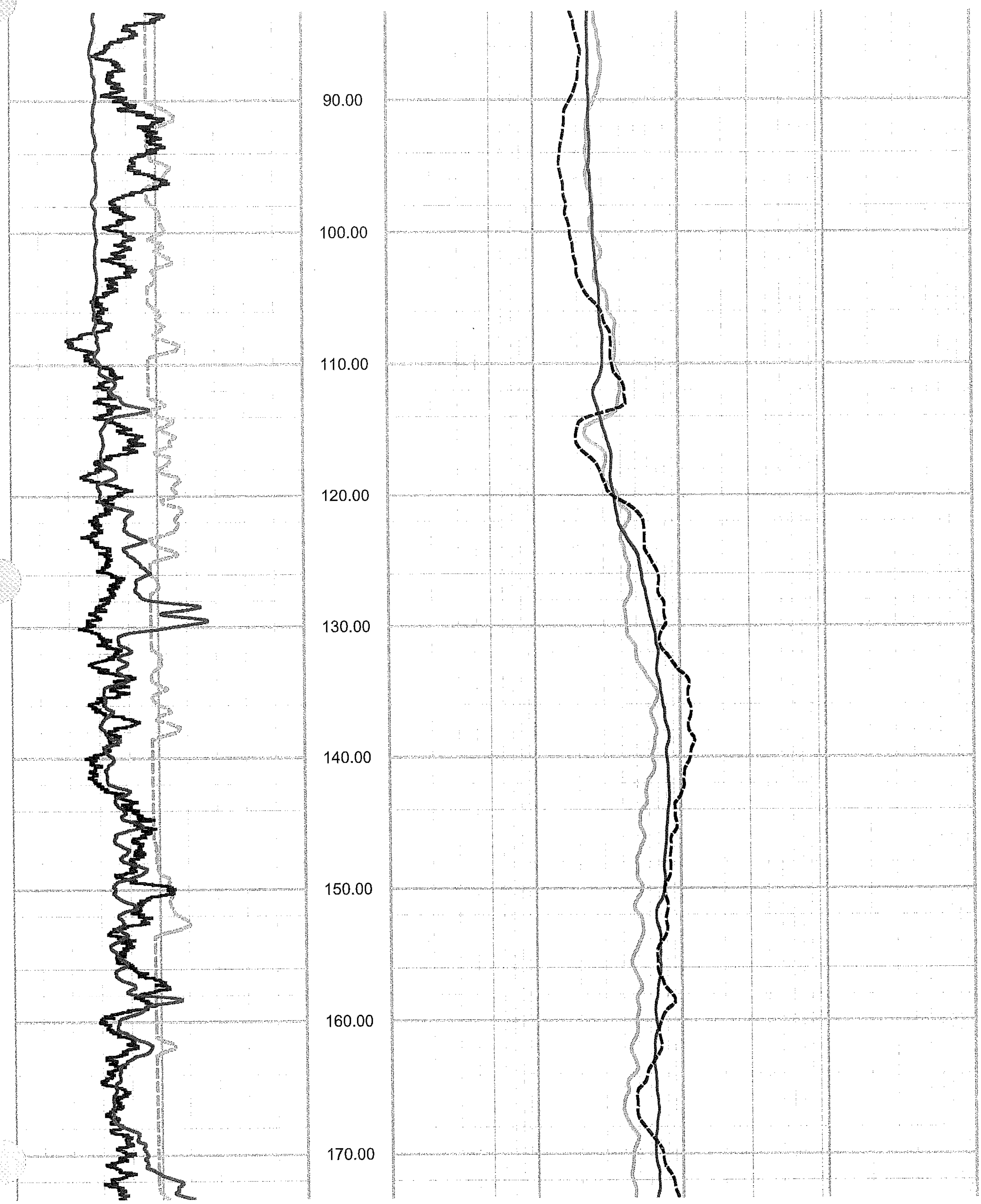
90.00
100.00
110.00
120.00
130.00
140.00
150.00
160.00
170.00

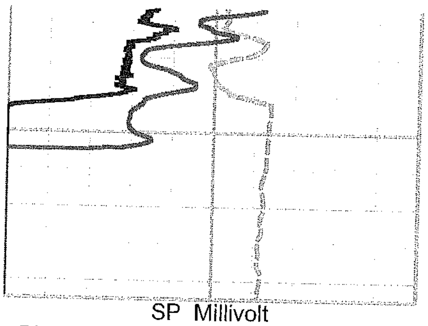


SP Millivolt
-500.00 500.00
NGAM CPS
0.00 100.00
CALP INCH
2.00 12.00

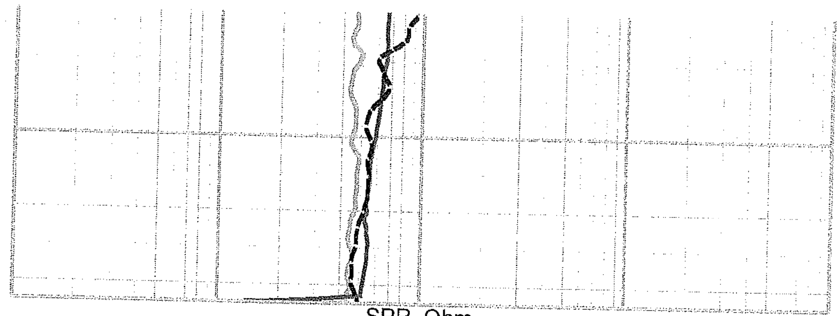
SHN Ohm M.
1.00 10000.00
LON Ohm M.
1.00 10000.00
SPR Ohm
1.00 10000.00


SP Millivolt
-500.00
500.00
NGAM CPS
0.00
100.00
CALP INCH
2.00
12.00
SPR Ohm
1.00
10000.00
LON Ohm M.
1.00
10000.00
SHN Ohm M.
1.00
10000.00
10.00
20.00
30.00
40.00
50.00
60.00
70.00
80.00


90.00
100.00
110.00
120.00
130.00
140.00
150.00
160.00
170.00



180.00



SP Millivolt
-500.00 500.00

NGAM CPS
0.00 100.00

CALP INCH
2.00 12.00

SPR Ohm
1.00 10000.00

LON Ohm M.
1.00 10000.00

SHN Ohm M.
1.00 10000.00

APPENDIX C

BORING GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION PROCEDURES AND CALIBRATION RECORDS

GEOVision SUSPENSION PS SEISMIC LOGGER/RECORDER

CALIBRATION PROCEDURE

Reviewed 7/21/08

Objective

The timing/sampling accuracy of seismic recorders or data loggers is required for several GEOVision field procedures including Seismic Refraction, Downhole P-S Seismic Velocity Logging, and Suspension P-S Seismic Velocity Logging. This procedure describes the method for measuring the timing accuracy of a seismic data logger, such as the OYO Model 170 or OYO/Robertson Model 3403. The objective of this procedure is to verify that the timing accuracy of the recorder is accurate to within 1%.

Frequency of Calibration

The calibration of each GEOVision seismic data logger is twelve (12) months. In the case of rented seismic logger/recorders, calibration must be performed prior to use.

Test Equipment Required

The following equipment is required. Item #2 must have current NIST traceable calibration.

1. Function generator, Krohn Hite 5400B or equivalent
2. Frequency counter, HP 5315A or equivalent
3. Test cables, from item 1 to item 2, and from item 1 to subject data logger.

Procedure

This procedure is designed to be performed using the accompanying Suspension P-S Seismic Logger/Recorder Calibration Data Form with the same revision number. All data must be entered and the procedure signed by the technician performing the test.

1. Record all identification data on the form provided.
2. Connect function generator to data logger (such as OYO Model 170) using test cable
3. Connect the function generator to the frequency counter using test cable.
4. Set signal generator to target frequency specified on data form, 0.25 volt (amplitude is approximate, modify as necessary to yield less than full scale waveforms on

logger display) peak sine wave. Verify frequency using the counter and note actual frequency on the data form.

5. Set data logger to file length specified on data form and record a data file to disk. Note file name on data form.

6. Measure the duration of 9 complete sine wave cycles on the data file. This measurement must be made using the analysis program PSLOG.EXE version 1.00, and saved as a .sps pick file. Note the duration in milliseconds in the spaces provided on the data form. Calculate average recorded sine wave frequency for each channel pair (Hn, Hr, V) by dividing the duration by 9. Note the average frequency of each channel pair on the data form.

7. Repeat steps 4 through 6 until all target frequencies have been recorded, producing 6 separate data and pick files.

Criteria

The average frequency for the nine cycles (obtained by dividing 9 cycles by the duration in seconds) must be within plus or minus 1% of the actual frequency for each of the 6 records.

If the results are outside this range, the data logger must be marked with a GEOVision REJECT tag until it can be repaired and retested.

If results are acceptable affix label indicating the initials of the person performing the calibration, the date of calibration, and the due date for the next calibration (12 months).

Procedure Approval

Approved by:

John G. Diehl	President
Name	Title
	July 21, 2008
Signature	Date

Calibration Laboratory Approval (if required):

Name	Title
Signature	Date

EDISON ESI℠

A SOUTHERN CALIFORNIA EDISON® Company

Calibration Report

TEST NUMBER
573795

Metrology
7300 Fenwick Lane
Westminster, CA 92683
Toll Free: 866-723-2257

GEOVision Geophysical Services
1124 Olympic Drive
Corona, CA 92881-3390

NVLAP®

Lab Code: 105014-0

Manufacturer: Oyo
Model Number: 3403
Description: Unit, Suspension Telemetry
Asset Number: 160024
Serial Number: 160024
Cal. Procedure: Customer
PO Number: 9200-090716-01

Ambient Temperature: 23° C
Ambient Humidity: 56% RH
Condition As Found: In Tolerance
Condition As Left: In Tolerance - No Adjustment
Calibration Date: 07/17/2009
Calibration Due Date: 07/17/2010
Calibration Interval: 12 Months

Remarks:
The unit was calibrated with the customer's procedure and specification's which have been reviewed by Metrology Engineering and documented in SCE Document M013987. The data can be found on pages 2 and 3 of this report with the original observation data on page 4.

Standards Utilized

I.D. No.	Manufacturer	Model No.	Description	Cal. Date	Due Date
S1-01252	Hewlett Packard	5335A OPT 010,203040	Counter, Universal	01/29/2009	07/29/2009
S1-01347	Hewlett Packard	3325A	Generator, Function, Synthesizer	05/04/2009	11/04/2009
S1-03686	Fluke	910	Standard, Frequency, Controlled, Gps	01/24/2009	01/24/2010

Calibration Performed By:				Quality Reviewer:	
Branson, Craig A CA3		Metrologist	714-895-0714	[signature]	7/17/09
Name		Title	Phone	Name	Date

Custom Specification Report

Test No. 573795
Asset No. 160024

Oyo 3403 Unit, Suspension Telemetry,

STEP NUM	FUNCTION TESTED	NOMINAL VALUE	AS FOUND	AS LEFT	Out of Tol	CALIBRATION TOLERANCE
	CH HN Frequency Sine Wave	50.00 Hz	50.00	Same		49.50 to 50.50 Hz [EMU 0.000250]
	|	100.0 Hz	100.0	Same		99.0 to 101.0 Hz [EMU 0.000500]
	|	200.0 Hz	200.2	Same		198.0 to 202.0 Hz [EMU 0.001000]
	|	500.0 Hz	500.0	Same		495.0 to 505.0 Hz [EMU 0.002500]
	|	1000 Hz	1000	Same		990 to 1010 Hz [EMU 0.005000]
	|	2000 Hz	2000	Same		1980 to 2020 Hz [EMU 0.010000]
	CH HR Frequency Sine Wave	50.00 Hz	50.00	Same		49.50 to 50.50 Hz [EMU 0.000250]
	|	100.0 Hz	100.0	Same		99.0 to 101.0 Hz [EMU 0.000500]
	|	200.0 Hz	200.0	Same		198.0 to 202.0 Hz [EMU 0.001000]
	|	500.0 Hz	500.0	Same		495.0 to 505.0 Hz [EMU 0.002500]
	|	1000 Hz	1001	Same		990 to 1010 Hz [EMU 0.005000]
	|	2000 Hz	2000	Same		1980 to 2020 Hz [EMU 0.010000]
	CH V Frequency Sine Wave	50.00 Hz	50.00	Same		49.50 to 50.50 Hz [EMU 0.000250]
	|	100.0 Hz	100.0	Same		99.0 to 101.0 Hz [EMU 0.000500]
	|	200.0 Hz	200.0	Same		198.0 to 202.0 Hz [EMU 0.001000]
	|	500.0 Hz	500.0	Same		495.0 to 505.0 Hz [EMU 0.002500]

Remarks:

MudCats CPM: Version 2.2.2 (Professional)
Src DUI: {9548AF3D-C74D-4C9F-AEEF-21EF560BC451} (c)
Doc DUI: {1269C0B2-3A13-416A-81BF-409D9887DDDA} (o)

ATTACHMENT 2
Page 1 of 2

Customer

Test No. 573795
Asset No. 160024

Custom Specification Report

Oyo 3403 Unit, Suspension Telemetry,

STEP NUM	FUNCTION TESTED	NOMINAL VALUE	AS FOUND	AS LEFT	Out of Tol	CALIBRATION TOLERANCE
	CH V Frequency Sine Wave	1000 Hz	1000	Same		990 to 1010 Hz [EMU 0.005000]
	|	2000 Hz	2000	Same		1980 to 2020 Hz [EMU 0.010000]

Remarks:

MudCats CPM: Version 2.2.2 (Professional)
Src DUI: {9548AF3D-C74D-4C9F-AEEF-21EF560BC451} (c)
Doc DUI: {1269C0B2-3A13-416A-81BF-409D9887DDDA} (o)

ATTACHMENT 2
Page 2 of 2

Customer

160024 Pg 4 of 4
~~160024~~ 7-17-09 CG
573795

GEOVision
geophysical services

SUSPENSION PS SEISMIC LOGGER/RECORDER CALIBRATION DATA FORM

INSTRUMENT DATA

System mfg.:	Oyo	Model no.:	3403
Serial no.:	160024	Calibration date:	7/17/2009
By:	Craig Branson	Due date:	7/17/2010
Counter mfg.:	Hewlett-Packard	Model no.:	5335A
Serial no.:	2626A09881	Calibration date:	1/29/2009
By:	SCE #S1-01252	Due date:	7/29/2009
Signal generator mfg.:	Hewlett-Packard	Model no.:	3325A
Serial no.:	2652A25647	Calibration date:	5/4/2009
By:	SCE #S1-01347	Due date:	11/4/2009

SYSTEM SETTINGS:

Gain:	8
Filter	10KHz
Range:	See sample period in table below
Delay:	0
Stack (1 std)	1
System date = correct date and time	7/17/2009 1037

PROCEDURE:

Set sine wave frequency to target frequency with amplitude of approximately 0.25 volt peak
Note actual frequency on data form.
Set sample period and record data file to disk. Note file name on data form.
Pick duration of 9 cycles using PSLOG.EXE program, note duration on data form, and save as .sps file. Calculate average frequency for each channel pair and note on data form.

Average frequency must be within +/- 1% of actual frequency at all data points.

Maximum error ((AVG-ACT)/ACT*100)% As found 0.10% As left 0.10%

Target Frequency (Hz)	Actual Frequency (Hz)	Sample Period (microS)	File Name	Time for 9 cycles Hn (msec)	Average Frequency Hn (Hz)	Time for 9 cycles Hr (msec)	Average Frequency Hr (Hz)	Time for 9 cycles V (msec)	Average Frequency V (Hz)
50.00	50.00	200	501	180.00	50.00	180.00	50.00	180.00	50.00
100.0	100.0	100	502	90.00	100.0	90.00	100.0	90.00	100.0
200.0	200.0	50	503	44.95	200.2	45.00	200.0	45.00	200.0
500.0	500.0	20	504	18.00	500.0	18.00	500.0	18.00	500.0
1000	1000	10	505	9.000	1000	8.990	1001	9.000	1000
2000	2000	5	506	4.500	2000	4.500	2000	4.500	2000

Calibrated by:	Craig Branson	7/17/2009	Craig Branson
	Name	Date	Signature
Witnessed by:	Robert Steller	7/17/2009	[signature]
	Name	Date	Signature

Suspension PS Seismic Recorder/Logger Calibration Data Form Rev 2.0 July 21, 2008

APPENDIX D

SHANNON & WILSON, INC.
SUBSURFACE CHARACTERIZATION

APPENDIX D

SHANNON & WILSON, INC.
SUBSURFACE CHARACTERIZATION

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TABLE

FIGURES

FIGURES (cont.)

APPENDIX D

SHANNON & WILSON, INC.
SUBSURFACE CHARACTERIZATION

D.1 INTRODUCTION

We reviewed the results of the explorations located within the general limits of the proposed Pontoon Casting Facility. We developed subsurface profiles using the results of the subsurface explorations presented in the contract documents.

The Geotechnical Data Report (GDR) contains the results of numerous in situ and laboratory tests. These tests include standard penetration tests (SPTs), cone penetration tests (CPTs), vane shear tests (VSTs), and pressuremeter testing (PMT). Laboratory tests include: one-dimensional consolidation tests, unconsolidated undrained (UU) and consolidated undrained (CU) triaxial tests, and direct simple shear (DSS) and cyclic direct simple shear tests. The soil classification and shear strength were compared using the results from the in situ and laboratory tests.

D.2 CONE PENETRATION TEST (CPT) INTERPRETATION

Thirty-seven CPTs were performed at the site by the Washington State Department of Transportation (WSDOT) and were included in the project's GDR. The raw CPT measurements were corrected to normalized parameters. The measured tip resistance was corrected with a cone factor of 0.8 and then adjusted for in situ stresses.

The normalized CPT parameters were then used by Shannon & Wilson in various empirical relationships to calculate soil behavior type index (Ic), soil behavior type (SBT), undrained shear strength, overconsolidation ratio (OCR), and permeability in general accordance with Lunne (1997) and Robertson (2009). The SBT is estimated using a step function to bin the data into generalized soil types. We estimated SBT with this traditional function and also using a continuous function that approximated the average step function to evaluate points that were close to the bin boundary (Robertson, 2009). The continuous-function SBT data was contoured along eight cross sections and is presented in Figures D-1 and D-2. The colors in these cross sections are set such that clean sands are yellow, silty sands are red, sandy silts and silts are green, and clays are blue. The numerical values shown on the cross-sections in Figures D-1 and D-2 are the Plasticity Indices determined from laboratory tests. The orientations of these subsurface cross sections are shown in Figure 3 in the main text of the report.

D.3 STANDARD PENETRATION TEST (SPT) INTERPRETATION

The SPTs were performed on 25 boreholes completed by WSDOT and 2 boreholes completed by Shannon & Wilson. SPT blow count results were corrected according to procedures in American Society of Civil Engineers (ASCE) 7 (i.e., Youd and others, 2001) for soils that classified by visual or CPT classification as silty sands to clean sands. The sandy soils were divided into an upper unit (from elevation -10 to -20 feet) and a lower unit that dips across the site (from elevation -37 to -90 feet). The upper sand unit was subdivided at an elevation of -17 feet based on the increase in SPT blow count. The lower sand SPT blow counts were further subdivided based on fines content above and below 35 percent. Figure D-3 presents all of the sandy SPT blow count data points versus depth. In addition, the mean and standard deviation over several depth ranges for SPT blow counts with fines content less than 35 percent are shown.

The blow counts measured during SPT sampling were normalized based on procedures from Youd and others (2001) to uniform $(N_1)_{60,CS}$ values. The parameters used in the normalization are presented below:

- Borehole Correction, C_B = 1.0 to 1.05 for various borehole diameters as shown in the exploration logs in Volume 1.
- Sampler Correction, C_S = 1.1 to 1.3 for samples taken without liners. Cs values vary and are based on iteratively calculated $(N_1)_{60,CS}$ values in accordance with recommendations from Idriss and Boulanger (2008).
- Energy Correction, C_E = 1.20 based on an average measured SPT energy of 72 percent.
- Fines Correction, C_F = 1.0
- Overburden Correction, $C_N = \frac{2.2}{1.2+\frac{\sigma'_{VO}}{Pa}} \leq 1.7$
- Rod Correction, C_R = 1.0 based on recommendations in Youd and others (2001), which indicate that the empirical database was evaluated without applying this factor.

D.4 VANE SHEAR TEST (VST) INTERPRETATION

The interpretation of undrained strength results from VSTs reported in the GDR was adopted in our analyses for evaluation of the subsurface conditions. Select VST undrained strength results are shown in Figures D-5 through D-41.

D.5 PRESSUREMETER TEST (PMT) INTERPRETATION

The PMT was performed at four boring locations for the project's GDR. The PMTs were performed at various depths, primarily in the cohesive soils. The PMT results were evaluated by In-Situ Tech, Inc. using three different soil models; Log Method, Load Model, and Unload Model. The results from the PMT Load and Unload Models are shown in Figures D-5 through D-41.

D.6 LABORATORY TEST INTERPRETATION

D.6.1 One-dimensional (1D) Consolidation

1D consolidation tests were completed by WSDOT. These tests were evaluated using a traditional Casagrande construction. Table D-1 presents the results of this evaluation, summarizing the estimations of recompression ratio (C_{re}), compression ratio (C_{ce}), past pressure, and OCR. The OCR is also presented in Figures D-5 through D-41 on a boring-by-boring basis.

D.6.2 Static Strength Testing

Various laboratory static strength tests were completed for the project's GDR including UU and CU triaxial tests, and DSS tests. The point at which shear strains increased at relatively constant shear stress was taken as the undrained strength of the sample. An approximate Stress History and Normalized Soil Engineering Parameter (SHANSEP) analysis was performed with the strength data (Ladd, 1974). It should be noted that the SHANSEP analysis was approximate, as the CU tests were not preformed in the SHANSEP framework of consolidating a sample beyond the past pressure and then unloading the sample to a desired OCR. To approximate the method, OCRs of the samples were assumed based on 1D consolidation tests. Results of this analysis indicated that using a Su/p' = 0.22 at an OCR =1 and an exponent (m) equal to 0.8, would estimate the lower bound strengths relative to the field and laboratory data.

D.6.3 Cyclic Direct Simple Shear

Cyclic direct simple shear tests were completed by WSDOT. These tests were used to evaluate the liquefaction susceptibility, cyclic resistance ratio (CRR), and post-cyclic monotonic undrained shear strength of silty soils. The number of cycles required to achieve the threshold criteria of an excess pore pressure ratio (Ru) of 0.9 and a cyclic shear strain of 4 percent at the various tested cyclic stress ratios are presented in Figure D-4. Also included in this figure, for comparison purposes, are typical response curves for sand consistent with ASCE 7. Several samples were monotonically sheared after cycling to evaluate the post-cyclic strength of the sample. The points at which the applied shear stress remained constant with increasing strain were taken as the post-cyclic strength. It should be noted that it appears that the stress and strain readings were zeroed before the monotonic shear, therefore requiring an adjustment to the post-cyclic strength and strain. The post-cyclic strengths are shown with the static strength test results in Figures D-5 through D-41. It should be noted that the cyclic loading of the silts results in an average reduction of strength of 25 percent at the end of cyclic testing.

D.7 DATA COMPARISONS

D.7.1 Cone Penetration Test (CPT) versus Visual Classification

A comparison of the SBT index from the CPT to the visual classification from nearby borings is shown in Figures D-5 through D-41. SBT values were assigned to visual classifications designations such as SP, SM/ML, or MH. The assigned values were varied until an approximate best fit match was achieved. The continuum of the visual classification designations is shown at the bottom of the plot. The purpose of this comparison was to evaluate the site-specific ability of the CPT to predict the visual Unified Soil Classification System (USCS) classification. It is well-published in literature that the SBT (or Ic) parameter of the CPT is not a good predictor of fines contentor the type of fines (silt or clay). This inability is noted in the clustering of the USCS classifications between ML and CH. However, the CPT appears to be a good predictor of cohesionless versus cohesive soil behavior. The CPT was able to identify cohesionless layers depicted in the boring logs and indicated others that likely exist between SPT samples. The relatively higher resolution of the CPT versus the SPT also allows for the thicknesses of these layers to be assessed. Based on this comparison, our engineering evaluations were based on interpreting soils in the "red" and "yellow" regions as silty sands, soils in the "green" region as silts with a PI<17, and soils in the "blue" region as medium to high plasticity silts and clays.

D.7.2 Plasticity Index

The plasticity index determined form Atterberg limit tests is plotted versus depth in Figures D-5 through D-41. Plasticity index data points are also shown in the SBT contour plots shown in Figures D-1 and D-2. Our interpretation of the distribution of these data points was that the PI<17 silts formed a boundary between the silty sands and medium- to high-plasticity silts. This layering is consistent with fluvial and overbank deposits.

D.7.3 Undrained Strength

A comparison of the undrained strengths from the laboratory tests, field tests, and CPT empirical relationships is shown in Figures D-5 through D-41. The undrained strengths interpreted from the laboratory and in situ tests are in good agreement with the PMT models and CPT empirical relationships. The results of all the strength tests and strength from calibration models for PMT's are summarized versus depth in Figure D-42.

D.7.4 Overconsolidation Ratio (OCR)

A comparison between the OCR interpreted from the 1D consolidation tests and from CPT empirical relationships are shown in Figures D-5 through D-41. The CPT OCR appears to show similar trends as the 1D consolidation tests. A macro view of the OCR versus depth indicates that the OCR is relatively constant versus depth. However, there are many large and small spikes in the OCR which are likely indicative of the periodic deposition of overbank soils.

D.8 REFERENCES

Idriss, I.M. and Boulanger, R.W., 2008, Soil liquefaction during earthquakes: Oakland, Calif., Earthquake Engineering Research Institute, MNO-12, 237 p.

Ladd, C. C. and Foott, Roger, 1974, New design procedure for stability of soft clays: Journal of the Geotechnical Engineering Division, v. 100, no. 7, p. 763-786.

Lunne, T., Robertson, P.K., and Powell, J.J.M., 1997, Cone penetration testing in geotechnical practice: Padstow, Cornwall, United Kingdom: T.J. International, 312 p.

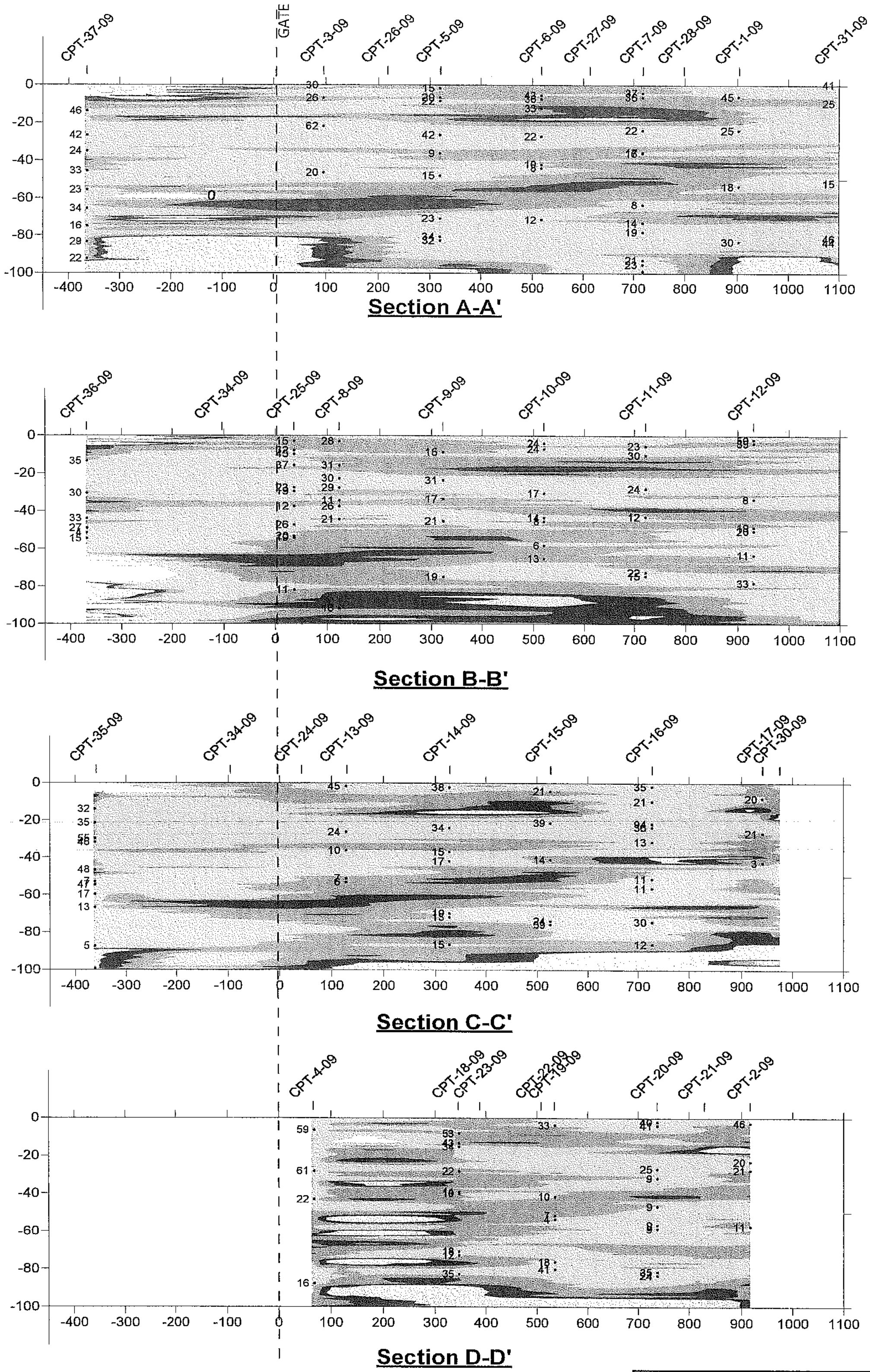
Robertson, P.K., 2009, Interpretation of cone penetration tests – a unified approach: Canadian Geotechnical Journal, v. 46, p. 1337-1355.

Youd, T. L., Idriss, I. M., Andrus, R. D., and others, 2001, Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils: Journal of Geotechnical and Geoenvironmental Engineering, v. 127, no. 10, p. 817-833.

TABLE D-1
CONSOLIDATION TEST RESULT SUMMARY

Boring	Sample	Sample Depth (ft)	Sample Elevation (ft)	Estimated Effective Overburden Stress σ'_V (psf)	USCS	Initial Moisture Content W_C (%)	Initial Moist Density γ_{WET} (pcf)	Initial Dry Density γ_D (pcf)	Initial Void Ratio e_0	Modified Normal Compression Index c_{ce}	Modified Recompression Index c_{re}	Estimated Pre-consolidation Pressure σ'_P (psf)	OCR Casagrande	Comments
H-01-08	S-1	46	-30	1970	ML	56	107	65	1.65	0.22	0.03	3410	1.73	
H-01-08	S-2	64	-48	2917	ML	46	125	72	1.41	0.19	0.03	7180	2.46	
H-04-08	S-1	36	-25	1337	MH	82	76	51	2.26	-	-	-	-	Data Inconclusive
H-04-08	S-2	96	-85	4373	ML	61	100	62	1.63	0.27	0.02	5830	1.33	
H-05P-09	S-7	20	-6	1175	OH	48	125	73	1.32	0.16	0.01	4800	4.09	
H-06P-09	S-10	28	-5	2052	OH	54	116	69	1.53	0.17	0.02	5080	2.48	
H-07P-09	S-7	18	-2	1260	OH	66	101	62	1.70	-	-	-	-	Data Inconclusive
H-07P-09	S-31	110	-94	5344	ML	43	138	78	1.12	0.16	0.01	9010	1.69	
H-11P-09	S-24	90	-73	4330	MH	52	115	68	1.33	-	-	-	-	Data Inconclusive
H-12P-09	S-6	18	-2	1266	OH	74	88	56	2.10	0.20	0.02	2060	1.63	
H-14P-09	S-24	83	-71	3709	ML	48	127	73	1.26	0.17	0.01	8460	2.28	
H-15P-09	S-25	90	-75	4214	MH	50	119	70	1.36	0.16	0.02	6330	1.50	
H-16-09	S-25	93	-76	4492	CH	59	109	66	1.57	0.23	0.02	7520	1.67	
H-18-09	S-9	25	-14	1141	MH	77	80	52	2.15	0.25	0.03	2460	2.16	
H-18-09	S-23	83	-72	3689	ML	53	116	69	1.54	0.17	0.01	5470	1.48	
H-20P-09	S-7	19	-2	1319	MH	77	83	54	2.08	-	-	-	-	Data Inconclusive
H-20P-09	S-21	74	-57	3467	MH	26	120	88	0.92	0.14	0.01	6350	1.83	
H-20P-09	S-27	99	-82	4782	MH	61	118	70	1.47	0.21	0.02	8800	1.84	
H-25-09	S-14	43	-24	2074	OH	76	83	54	2.10	0.28	0.03	3720	1.79	
H-25-09	S-20	70	-51	3359	MH	51	120	70	1.38	0.16	0.01	6070	1.81	
H-26-09	S-6	18	-6	979	MH	64	99	61	1.73	0.17	0.01	4050	4.14	
H-28-09	S-9	25	-14	1083	OH	85	76	50	2.45	-	-	-	-	Data Inconclusive
H-29P-09	S-7	20	-7	1124	OH	72	88	56	1.84	-	-	-	-	Data Inconclusive
H-31P-09	S-6	18	-6	969	OH	85	76	51	2.32	0.29	0.04	1720	1.78	
H-36-09	S-4	13	-1	825	OH	76	87	56	1.99	0.22	0.02	2210	2.68	
H-38-09	S-6	18	-2	1239	MH	54	114	68	1.48	0.15	0.01	2960	2.39	
H-40-09	S-4	13	3	1023	OH	91	72	48	2.39	0.22	0.03	1700	1.66	
H-47-09	S-6	18	-2	1250	OH	62	101	62	1.69	0.19	0.02	2920	2.34	

SOUTH NORTH



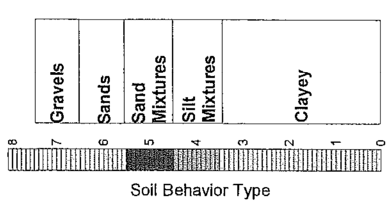


FIG. D-1

SR 520 Pontoon Casting Facility
Aberdeen, Washington

SUMMARY OF IDEALIZED SOIL BEHAVIOR NORTH-SOUTH SECTIONS

August 2010 21-1-21190-015

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. D-1

TABLE D-1
CONSOLIDATION TEST RESULT SUMMARY

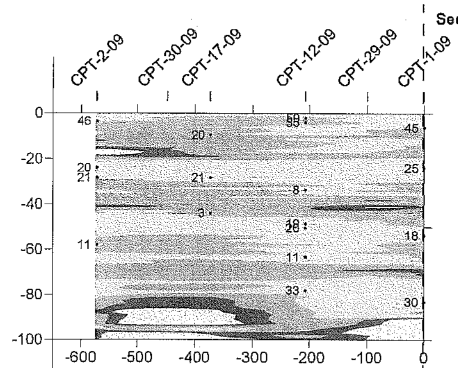
Boring	Sample	Sample Depth (ft)	Sample Elevation (ft)	Estimated Effective Overburden Stress σ'_V (psf)	USCS	Initial Moisture Content w_C (%)	Initial Moist Density γ_{WET} (pcf)	Initial Dry Density γ_D (pcf)	Initial Void Ratio e_0	Modified Normal Compression Index c_{ce}	Modified Recompression Index c_{re}	Estimated Pre-consolidation Pressure σ'_P (psf)	OCR Casagrande	Comments
H-49-09	S-6	18	-3	1148	OH	55	111	67	1.51	0.14	0.02	3420	2.98	
H-50-09	S-4	25	-13	1222	OH	87	75	50	2.28	0.22	0.03	2370	1.94	
H-52-09	S-6	18	-6	975	OH	96	59	42	2.45	-	-	-	-	Data Inconclusive

Notes:
OCR = overconsolidation ratio
pcf = pounds per cubic foot
psf = pounds per square fot
USCS = Unified Soil Classification System

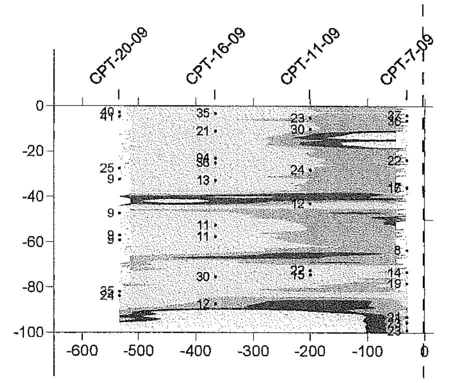
Soil Behavior Type Figures.xlsm Printed: 8/23/2010 11:05 AM

jnb/mdg

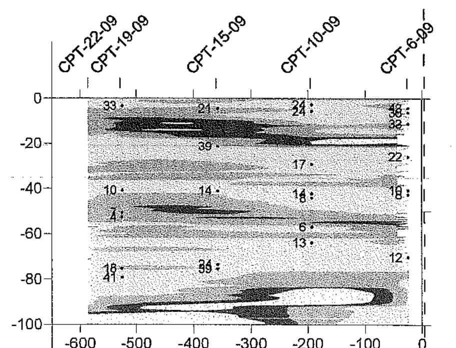
EAST | Section A-A' | WEST



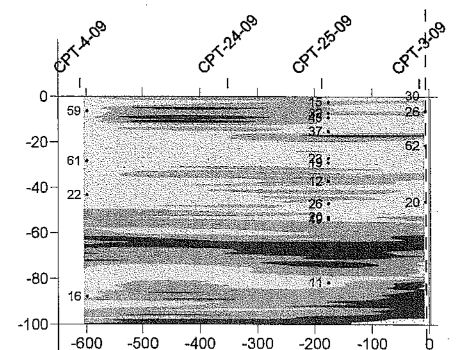
Section E-E''



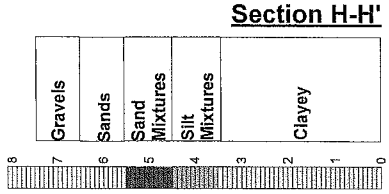
Section F-F''



Section G-G''



Section H-H'



Soil Behavior Type

SR 520 Pontoon Casting Facility
Aberdeen, Washington

**SUMMARY OF IDEALIZED SOIL BEHAVIOR
EAST-WEST SECTIONS**

August 2010 | 21-1-21190-015

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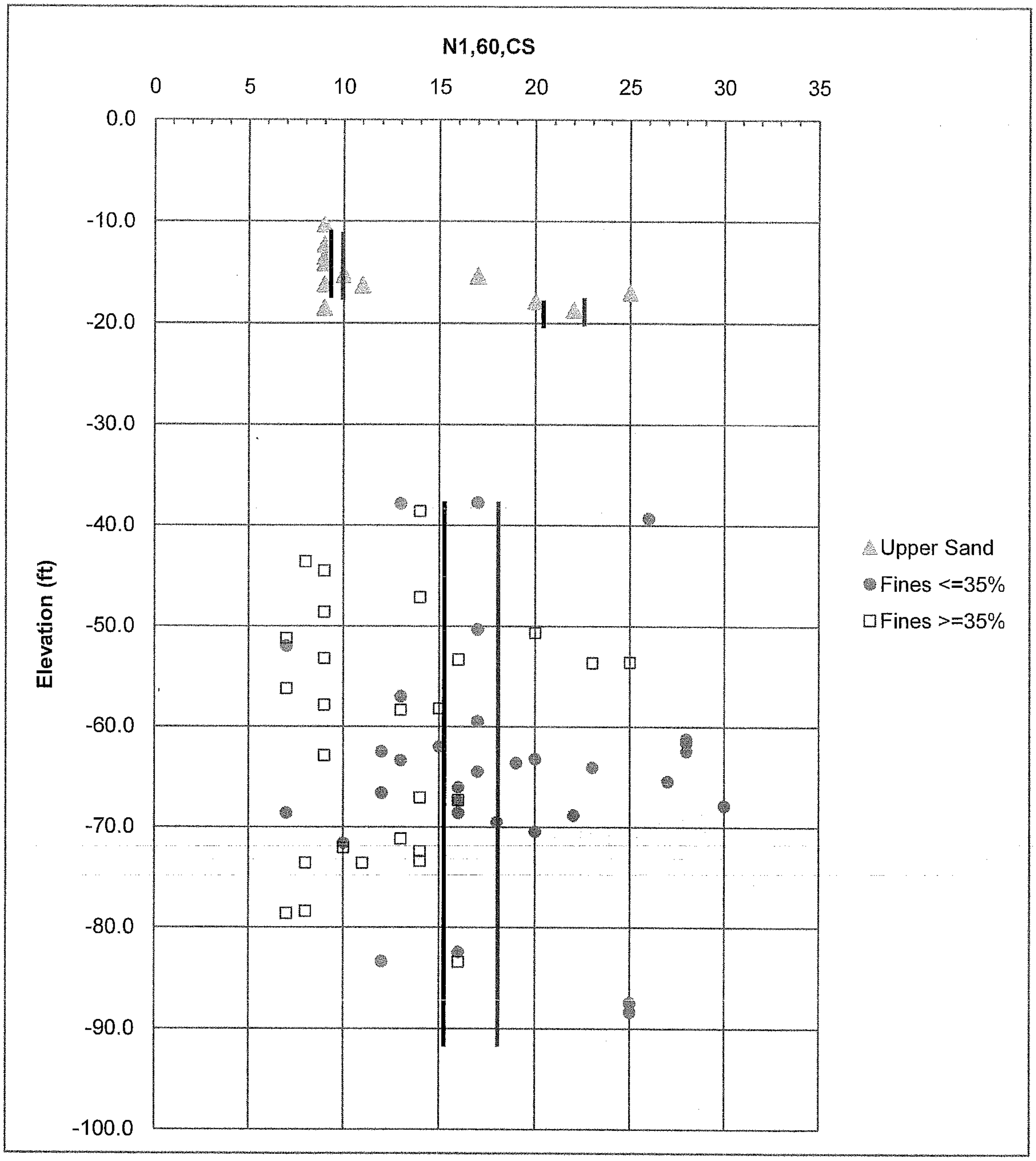
FIG. D-2

FIG. D-2



NOTES

1. Red lines represent the mean of the data. Black lines represent the blowcount assigned in the numerical models. Samples with fines contents greater than 35% were not used in the calculation of the mean.

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N1,60 BLOW COUNTS IN SAND UNITS

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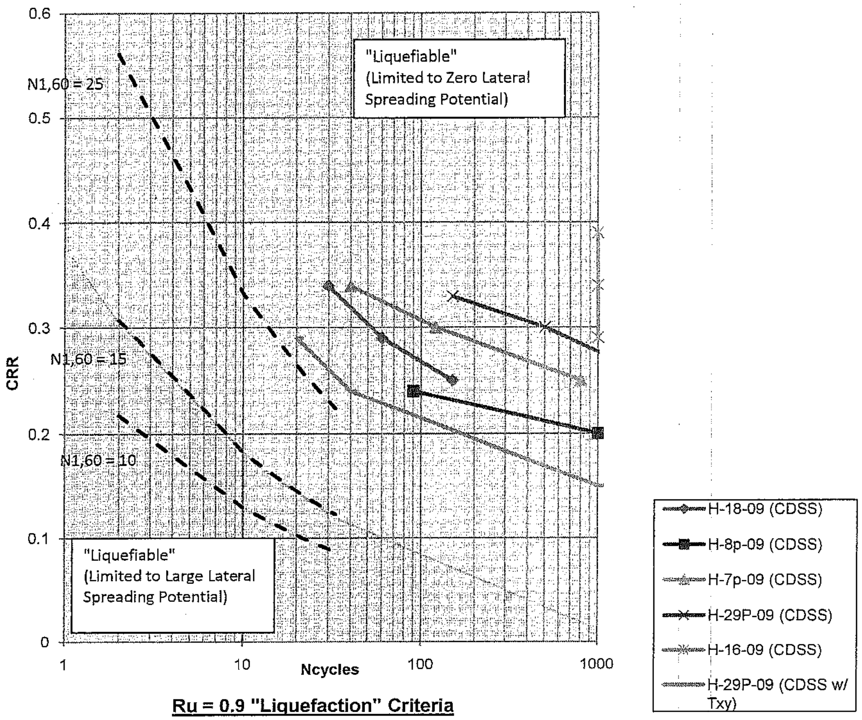
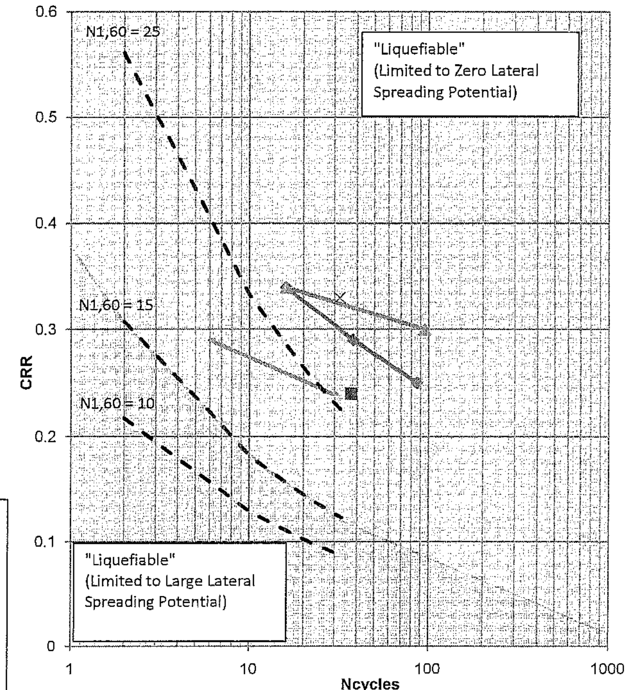
FIG. D-3



Ru = 0.9 "Liquefaction" Criteria

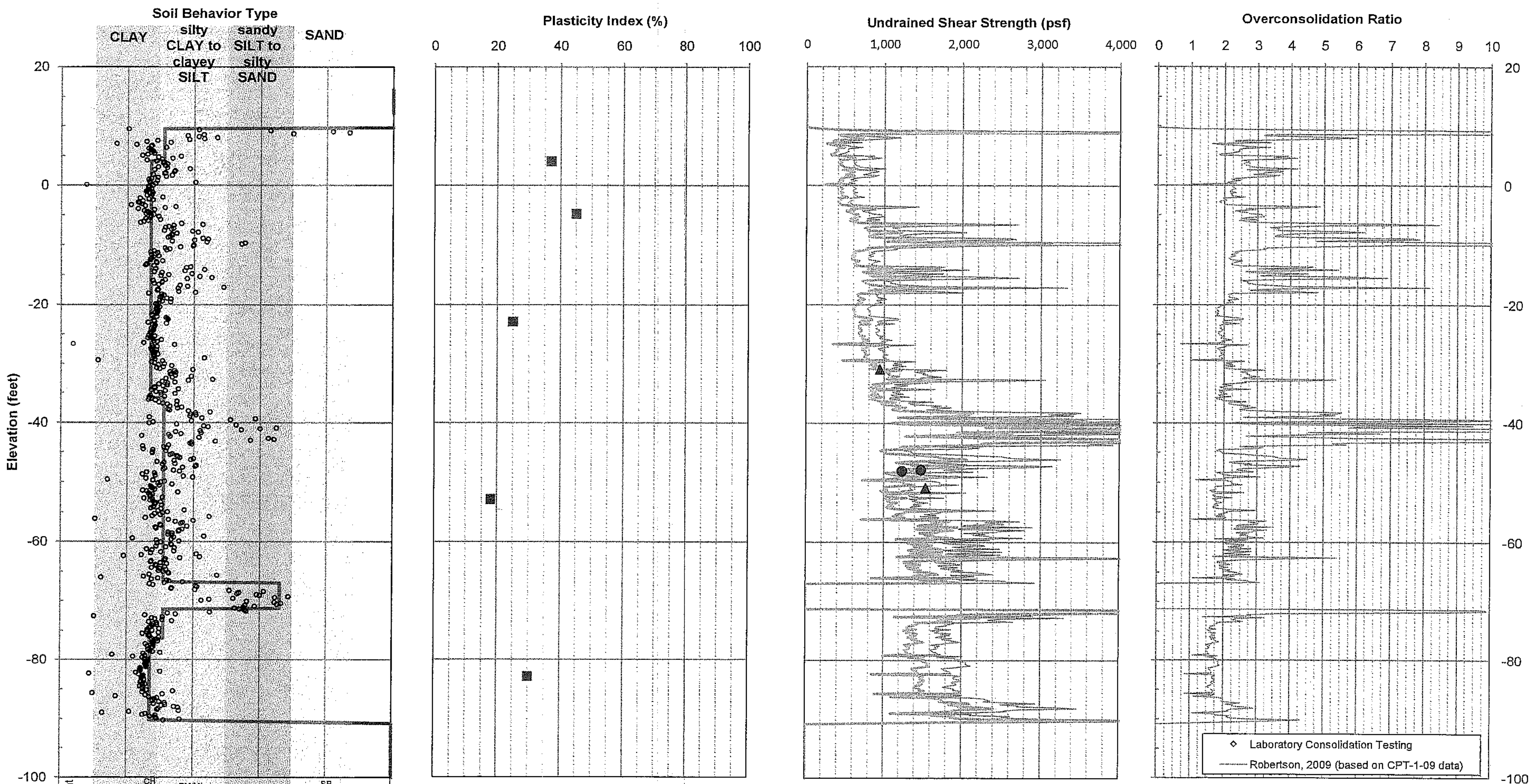


4% Shear Strain Criteria

NOTES

1. The number of uniform cycles at a given CSR are plotted based on the indicated "liquefaction criteria".

SR 520 Pontoon Casting Facility Aberdeen, Washington	
CDSS INTERPRETATION CRR VS NCYCLES	
August 2010	21-1-21190-016
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. D-4**



○ Robertson, 2009 (based on CPT-1-09 data)

H-01-08 Soil Boring Log

◇ Laboratory Consolidation Testing

Robertson, 2009 (based on CPT-1-09 data)

- Consolidated-Undrained Triaxial Test
- Unconsolidated-Undrained Triaxial Test
- ▲ In-Situ Vane Shear Test
- ◆ Pressuremeter Test (Loading)
- ◇ Pressuremeter Test (Unloading)
- Static Direct Simple Shear Test
- Post-Cyclic Direct Simple Shear Test
- Robertson, 2009 (based on CPT-1-09 data)
- Ladd and Foott, 1974 (based on CPT OCR estimates)

NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-01-08, which is in relatively close proximity to CPT-1-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-1-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-1-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

SR 520 Pontoon Casting Facility
Aberdeen, Washington

SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-1-09 AND BORING H-01-08

February 2011 21-1-21190-015

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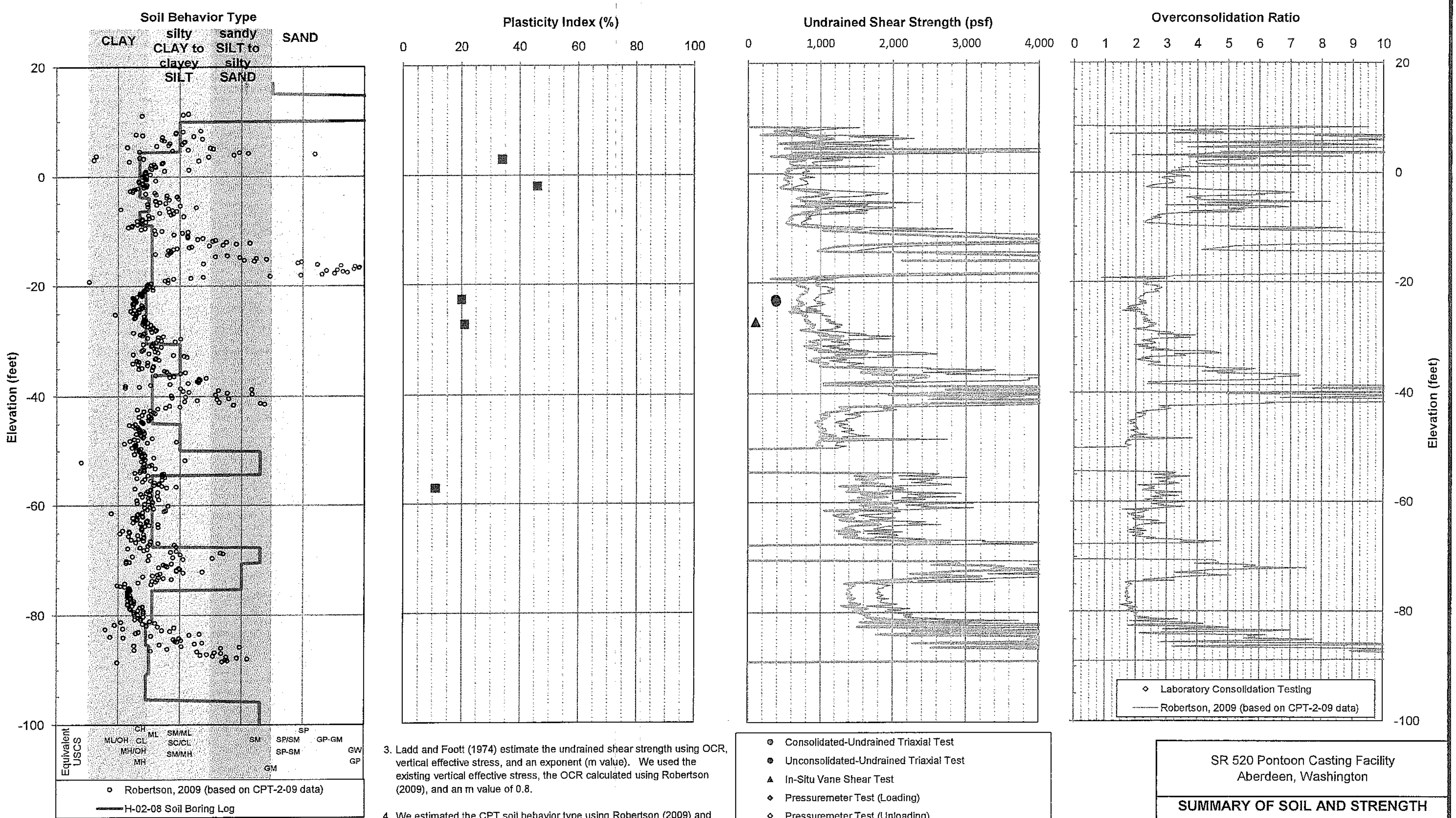
FIG. D-5



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-02-08, which is in relatively close proximity to CPT-2-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-2-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-2-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

SR 520 Pontoon Casting Facility
Aberdeen, Washington

SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-2-09 AND BORING H-02-08

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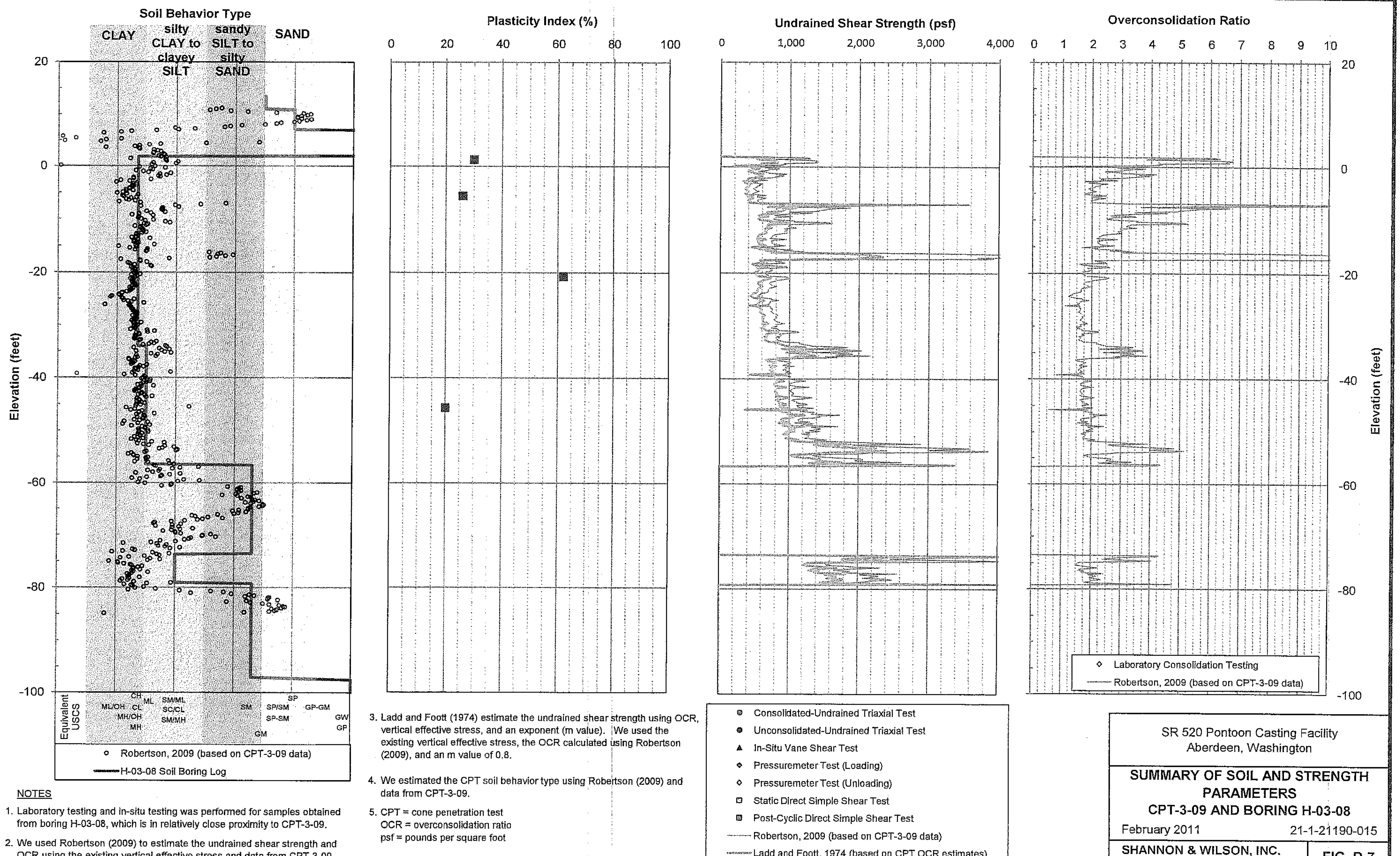
FIG. D-6



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-03-08, which is in relatively close proximity to CPT-3-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-3-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-3-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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Aberdeen, Washington

SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-3-09 AND BORING H-03-08

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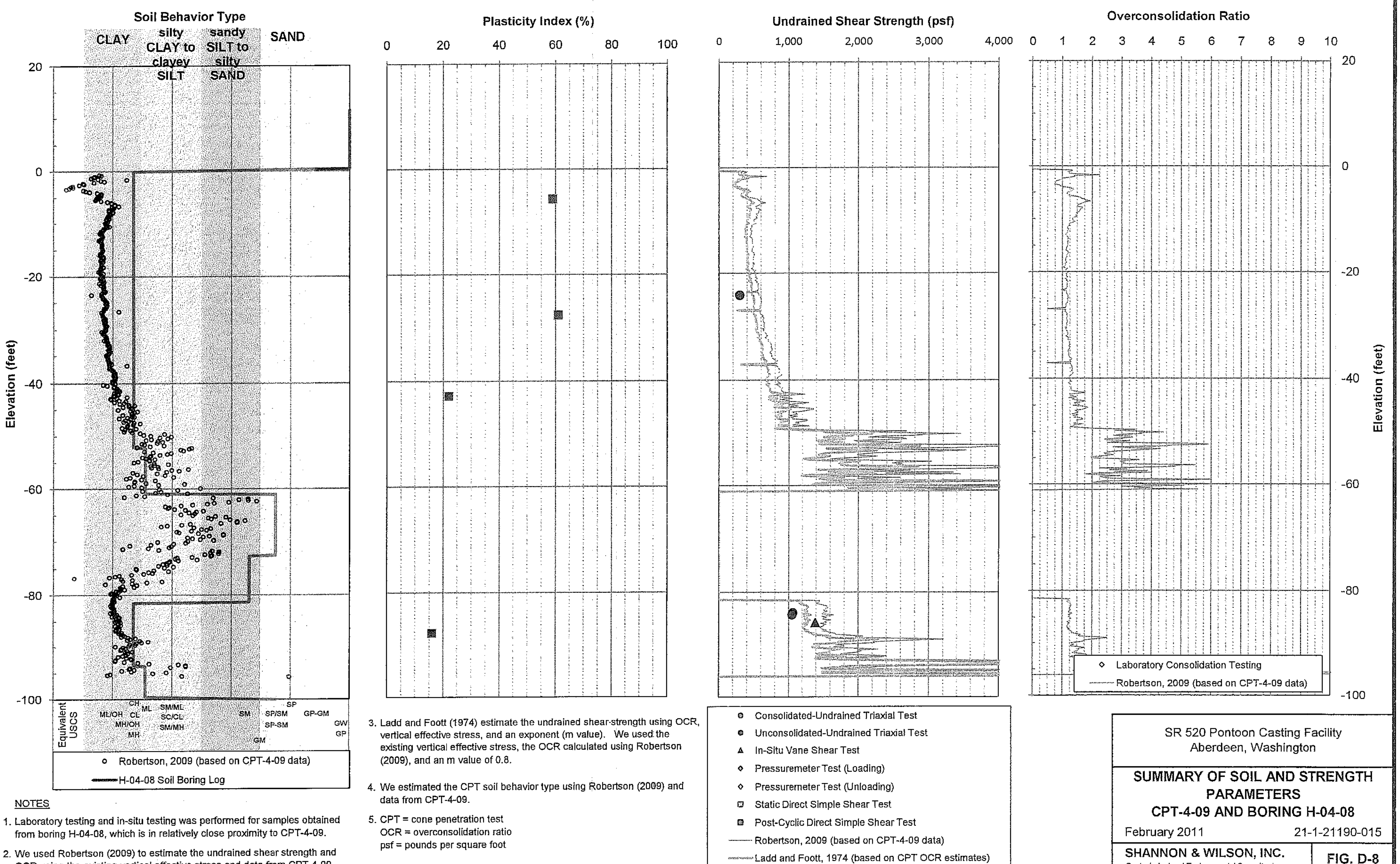
FIG. D-7

Summary of CPT Data.xlsm Printed: 2/15/2011 2:39 PM



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-04-08, which is in relatively close proximity to CPT-4-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-4-09.
3. Ladd and Foott (1974) estimate the undrained shear-strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-4-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS CPT-4-09 AND BORING H-04-08

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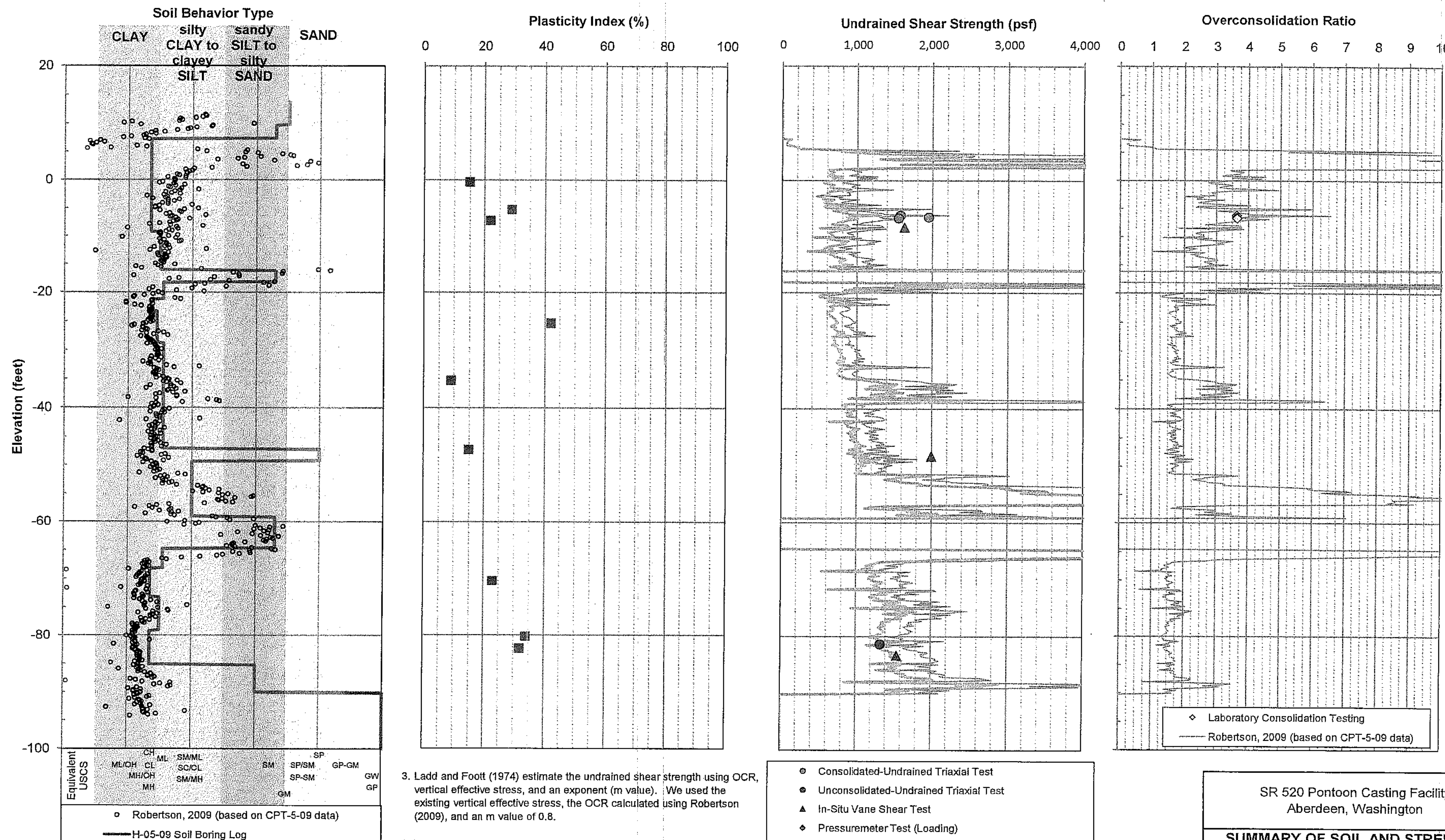
FIG. D-8



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-05-09, which is in relatively close proximity to CPT-5-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-5-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-5-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

- Consolidated-Undrained Triaxial Test
- Unconsolidated-Undrained Triaxial Test
- In-Situ Vane Shear Test
- Pressuremeter Test (Loading)
- Pressuremeter Test (Unloading)
- Static Direct Simple Shear Test
- Post-Cyclic Direct Simple Shear Test
- Robertson, 2009 (based on CPT-5-09 data)
- Ladd and Foott, 1974 (based on CPT OCR estimates)

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-5-09 AND BORING H-05-09

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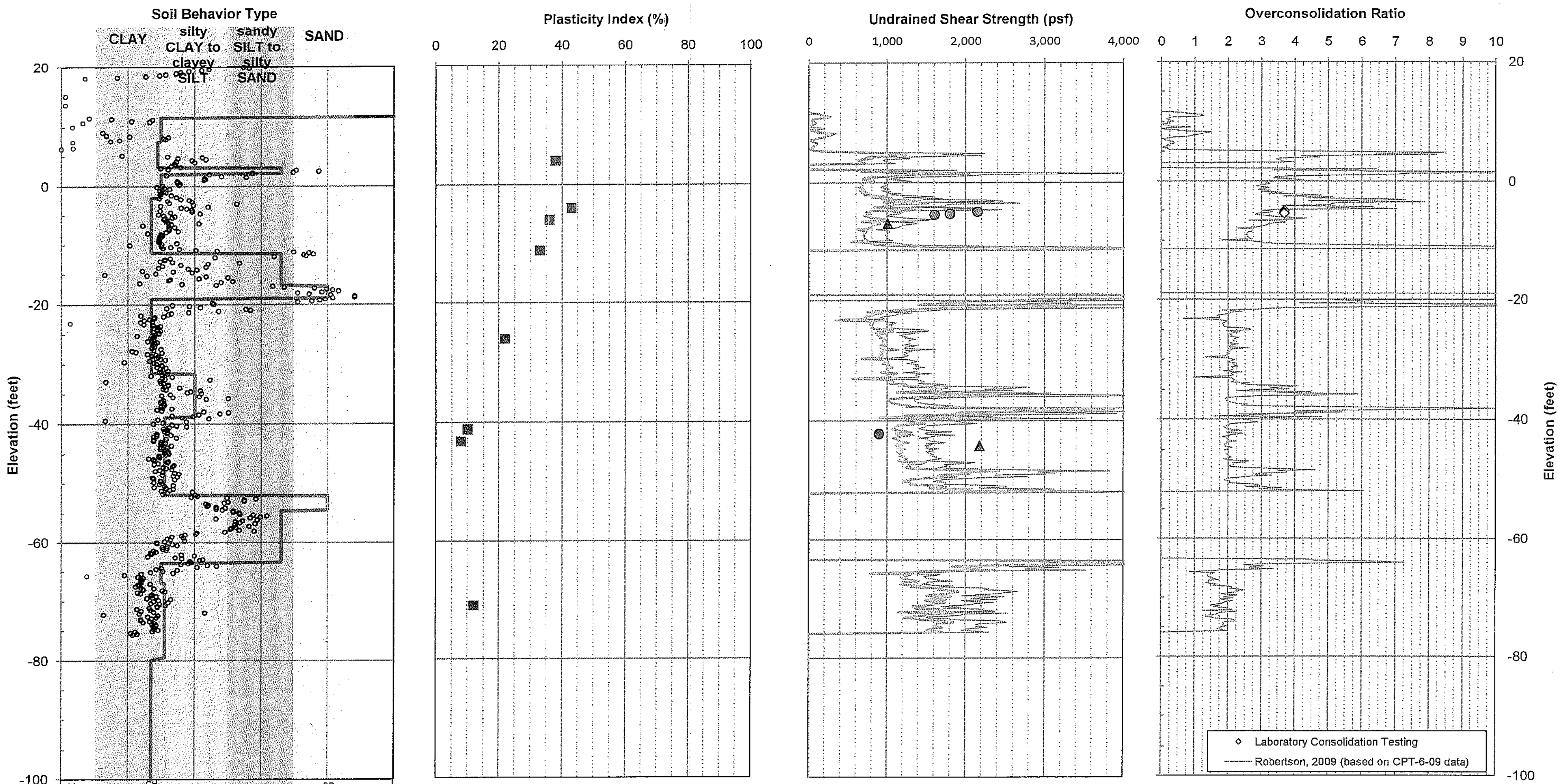
FIG. D-9

Summary of CPT Data.xlsm Printed: 2/15/2011 2:40 PM



Robertson, 2009 (based on CPT-6-09 data)
H-06-09 Soil Boring Log

Consolidated-Undrained Triaxial Test
Unconsolidated-Undrained Triaxial Test
In-Situ Vane Shear Test
Pressuremeter Test (Loading)
Pressuremeter Test (Unloading)
Static Direct Simple Shear Test
Post-Cyclic Direct Simple Shear Test
Robertson, 2009 (based on CPT-6-09 data)
Ladd and Foott, 1974 (based on CPT OCR estimates)

NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-06-09, which is in relatively close proximity to CPT-6-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-6-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-6-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-6-09 AND BORING H-06-09

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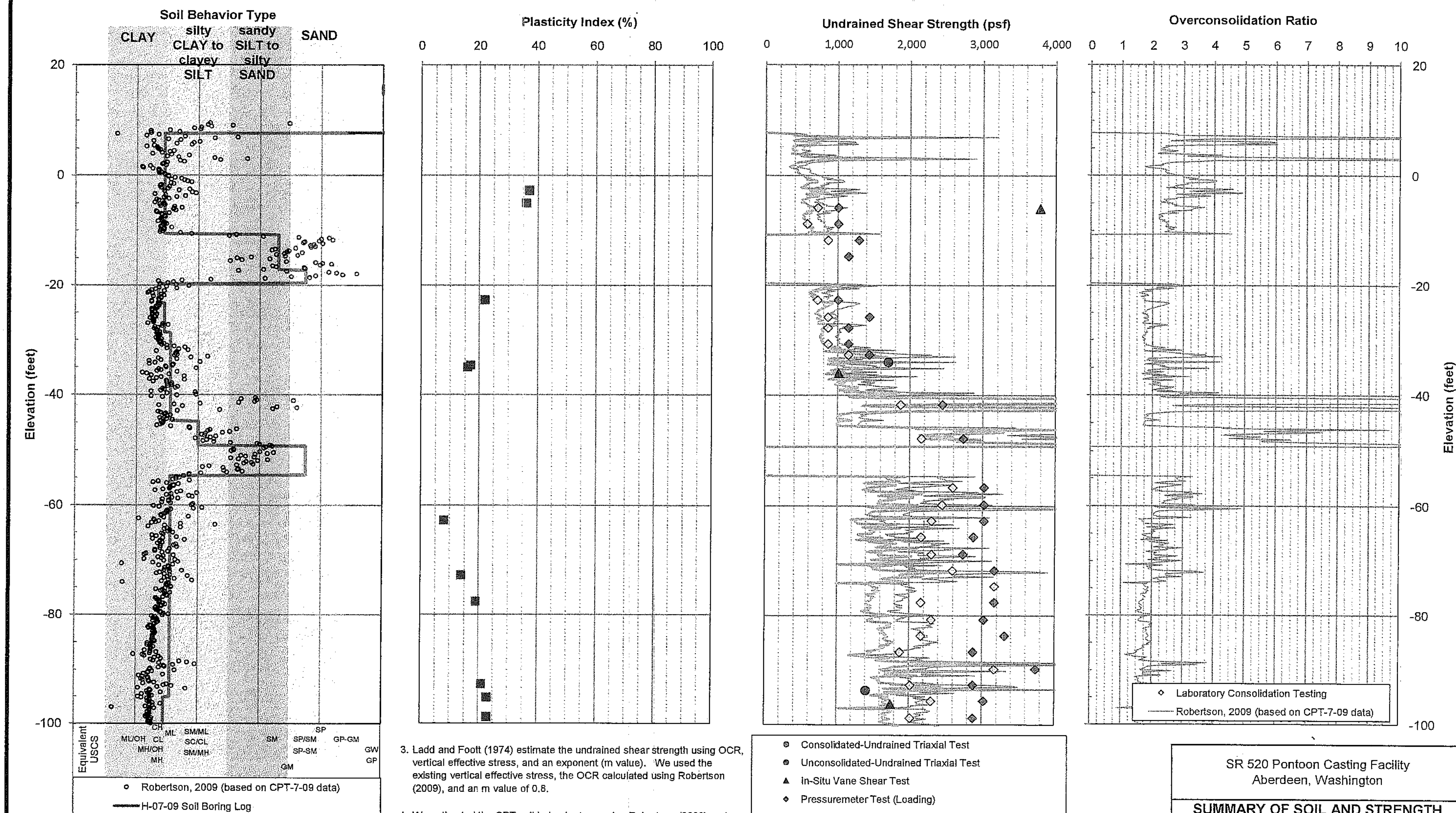
FIG. D-10



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-07-09, which is in relatively close proximity to CPT-7-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-7-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-7-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

- Consolidated-Undrained Triaxial Test
- Unconsolidated-Undrained Triaxial Test
- In-Situ Vane Shear Test
- Pressuremeter Test (Loading)
- Pressuremeter Test (Unloading)
- Static Direct Simple Shear Test
- Post-Cyclic Direct Simple Shear Test
- Robertson, 2009 (based on CPT-7-09 data)
- Ladd and Foott, 1974 (based on CPT OCR estimates)

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-7-09 AND BORING H-07-09

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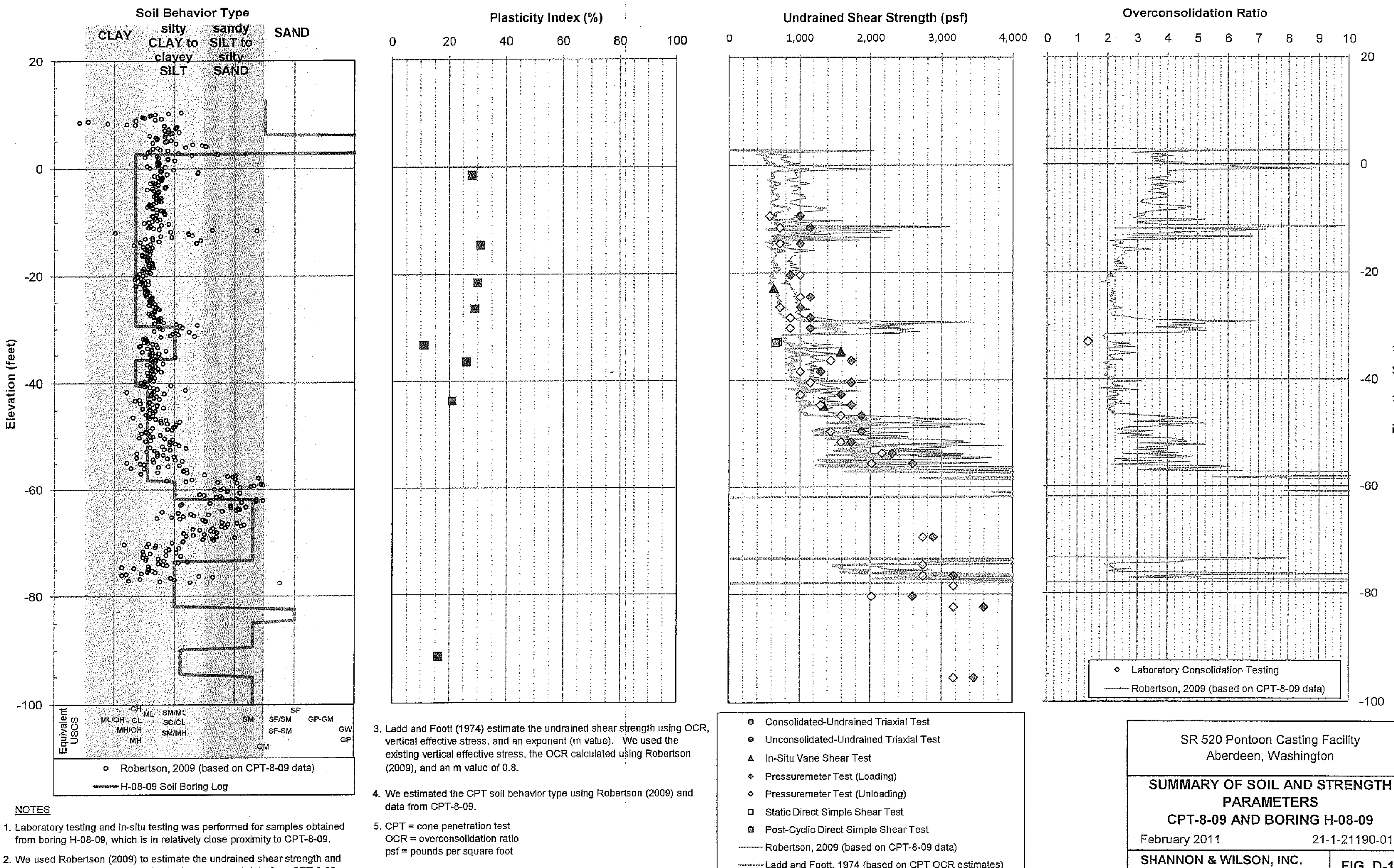
FIG. D-11



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-08-09, which is in relatively close proximity to CPT-8-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-8-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-8-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-8-09 AND BORING H-08-09

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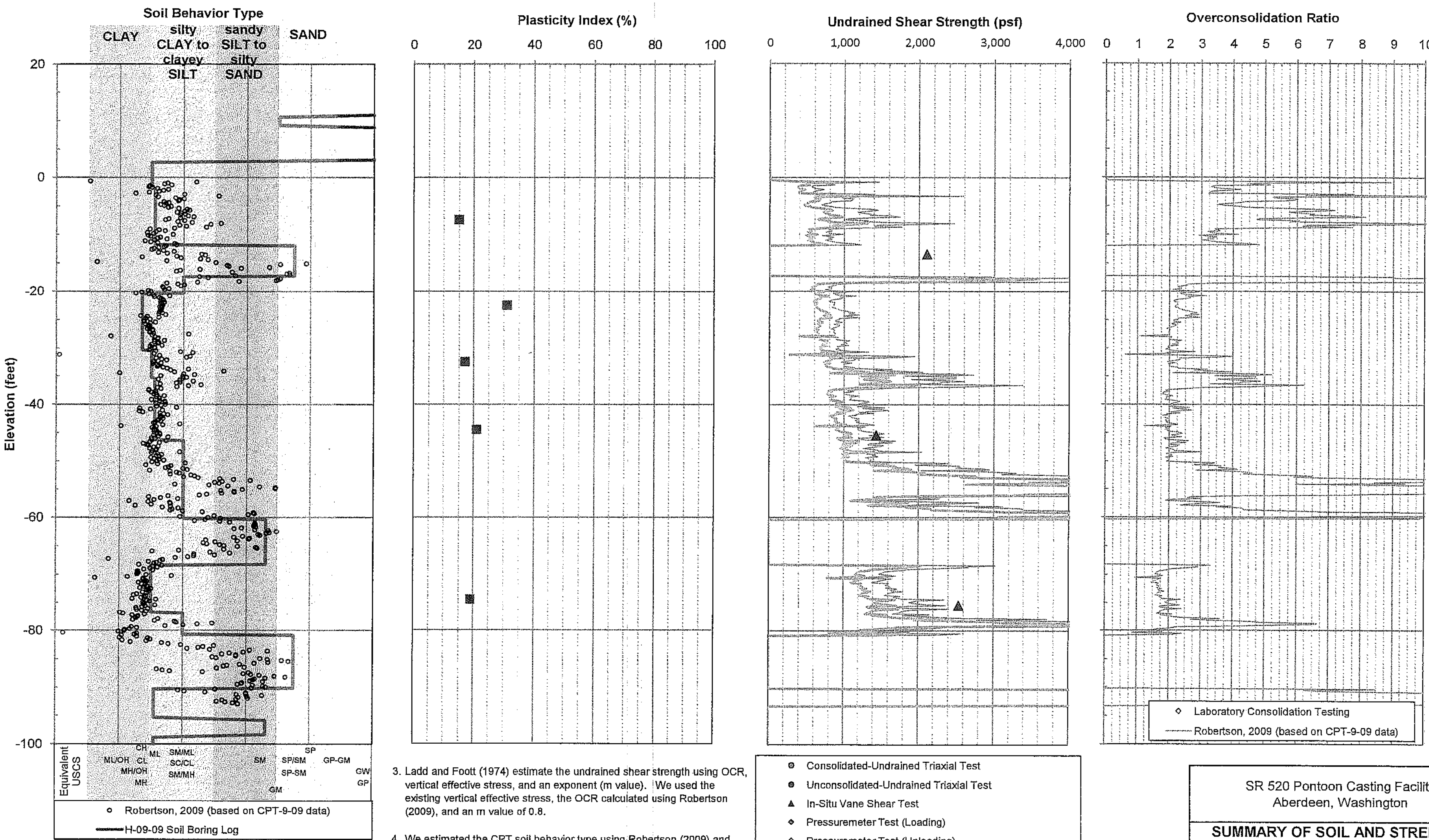
FIG. D-12



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-09-09, which is in relatively close proximity to CPT-9-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-9-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-9-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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Aberdeen, Washington

SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-9-09 AND BORING H-09-09

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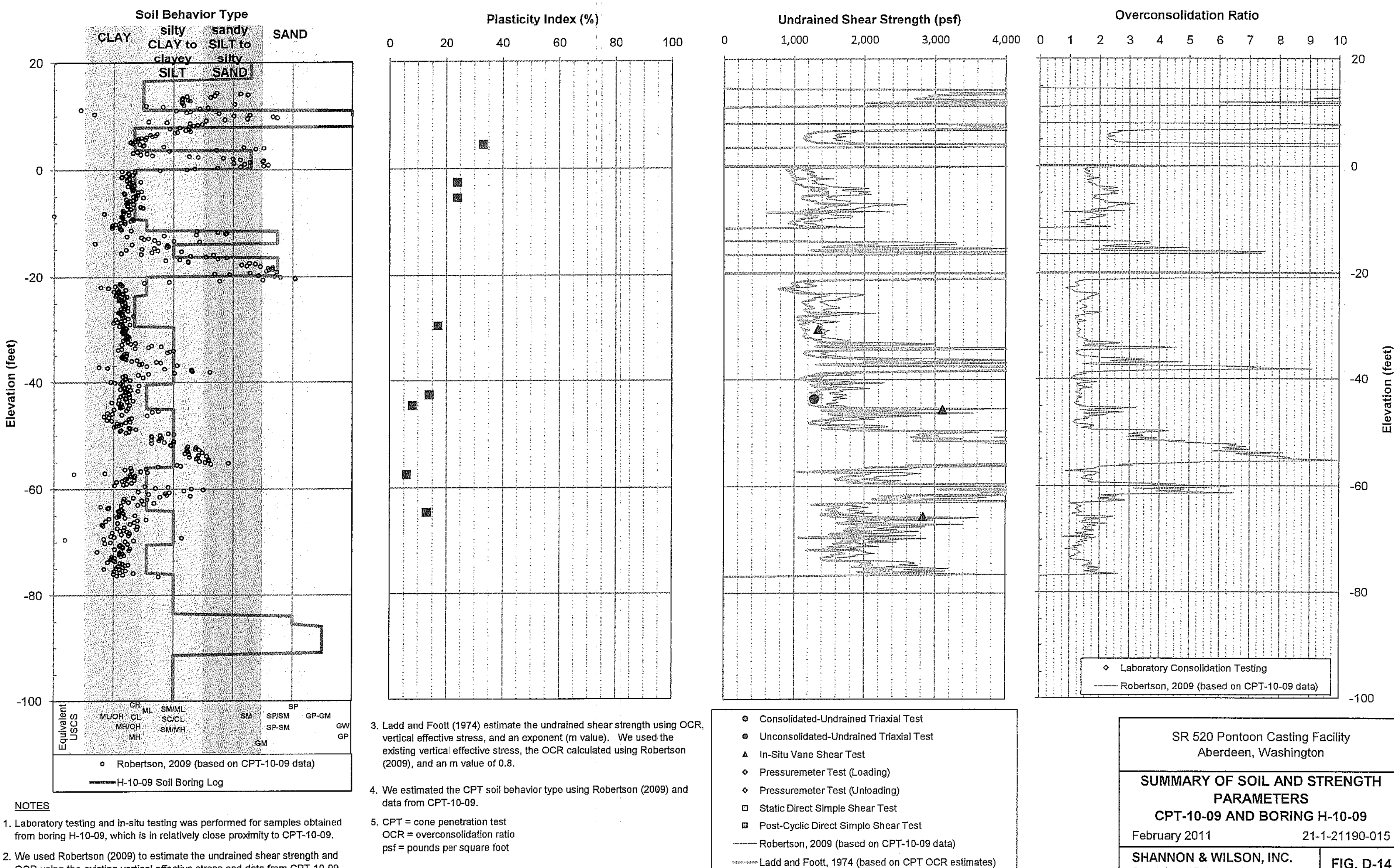
FIG. D-13

Summary of CPT Data.xlsm Printed: 2/15/2011 2:40 PM



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-10-09, which is in relatively close proximity to CPT-10-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-10-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-10-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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Aberdeen, Washington

SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-10-09 AND BORING H-10-09

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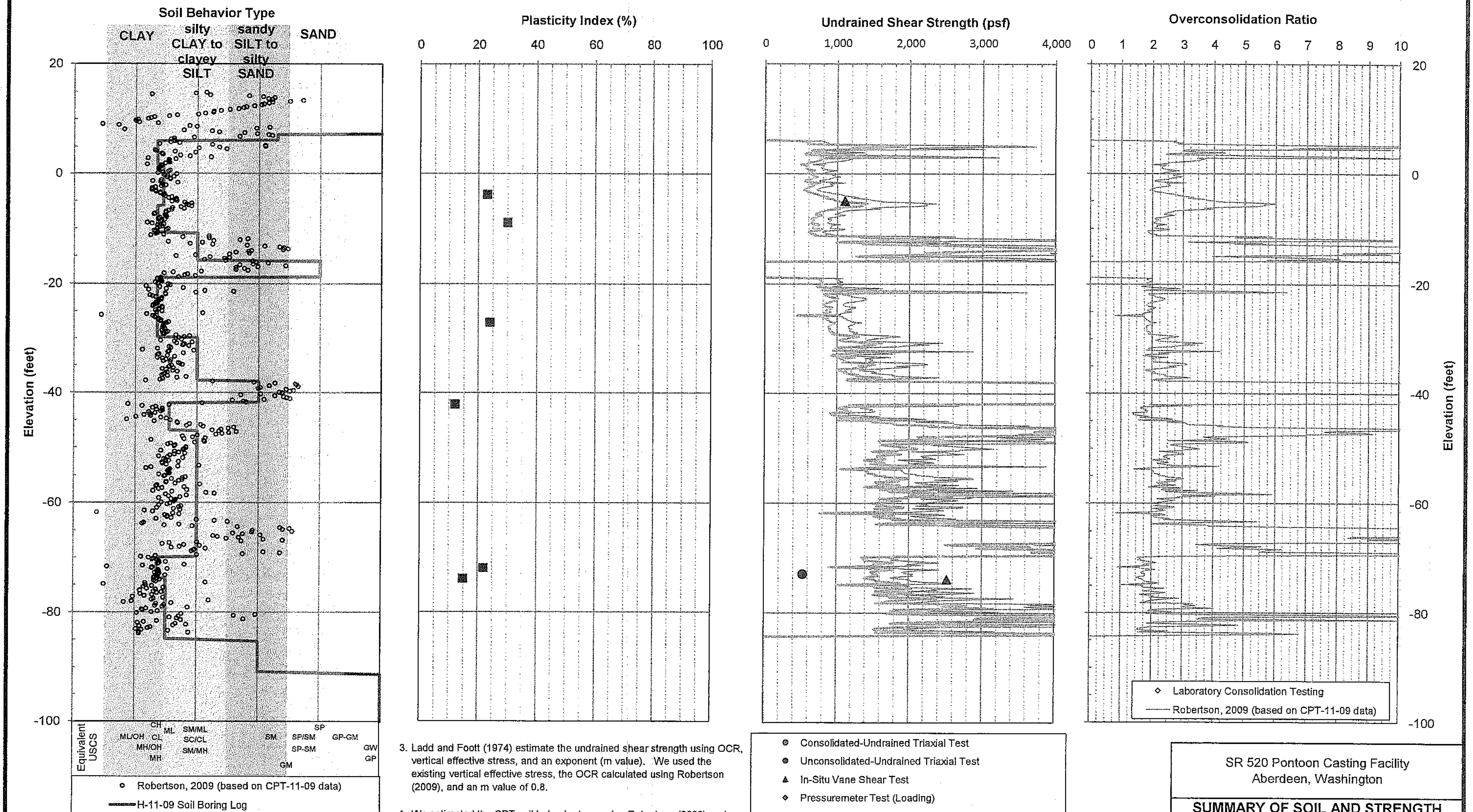
FIG. D-14



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-11-09, which is in relatively close proximity to CPT-11-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-11-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-11-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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Aberdeen, Washington

SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-11-09 AND BORING H-11-09

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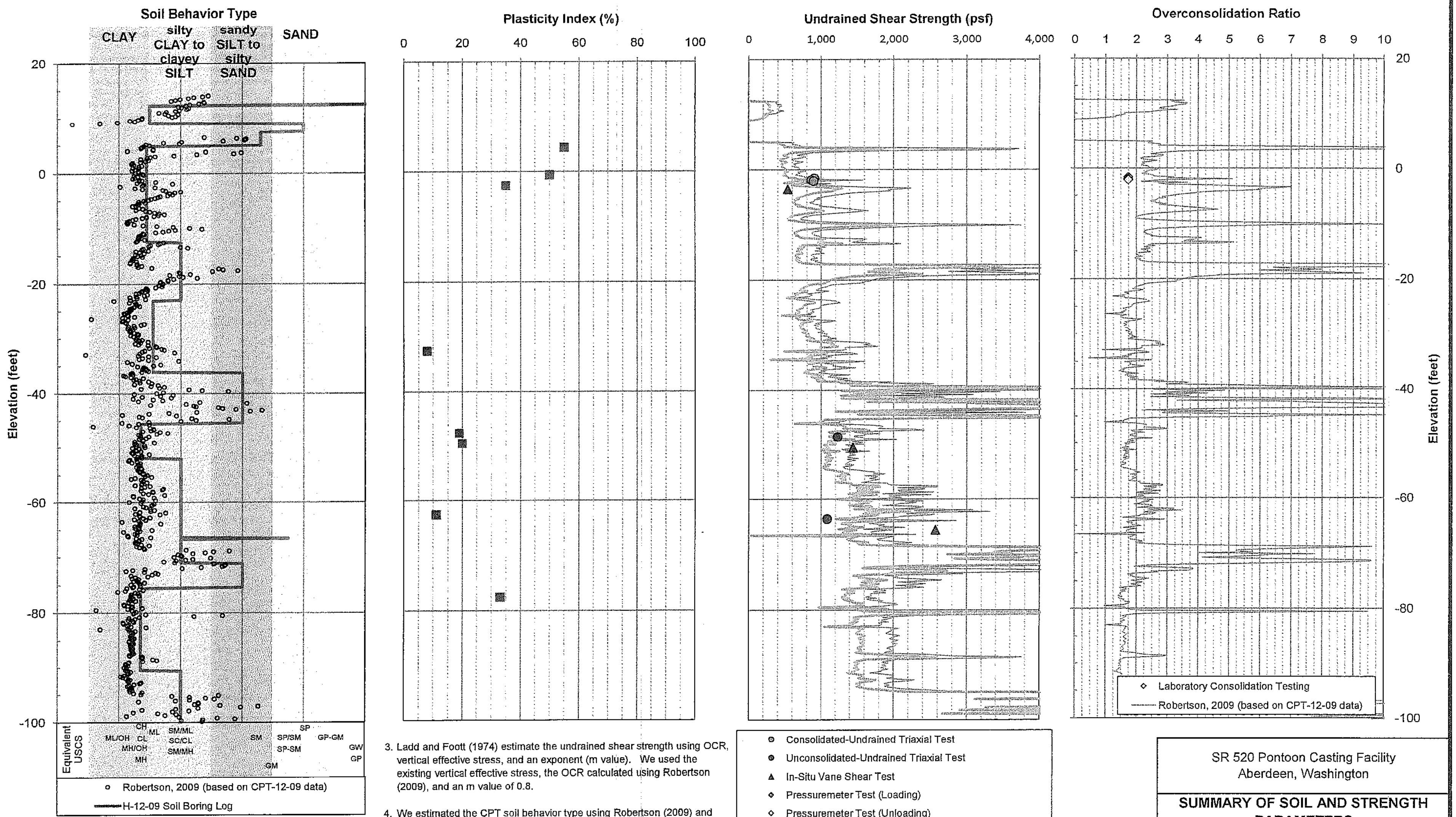
FIG. D-15



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-12-09, which is in relatively close proximity to CPT-12-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-12-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-12-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

SR 520 Pontoon Casting Facility
Aberdeen, Washington

SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-12-09 AND BORING H-12-09

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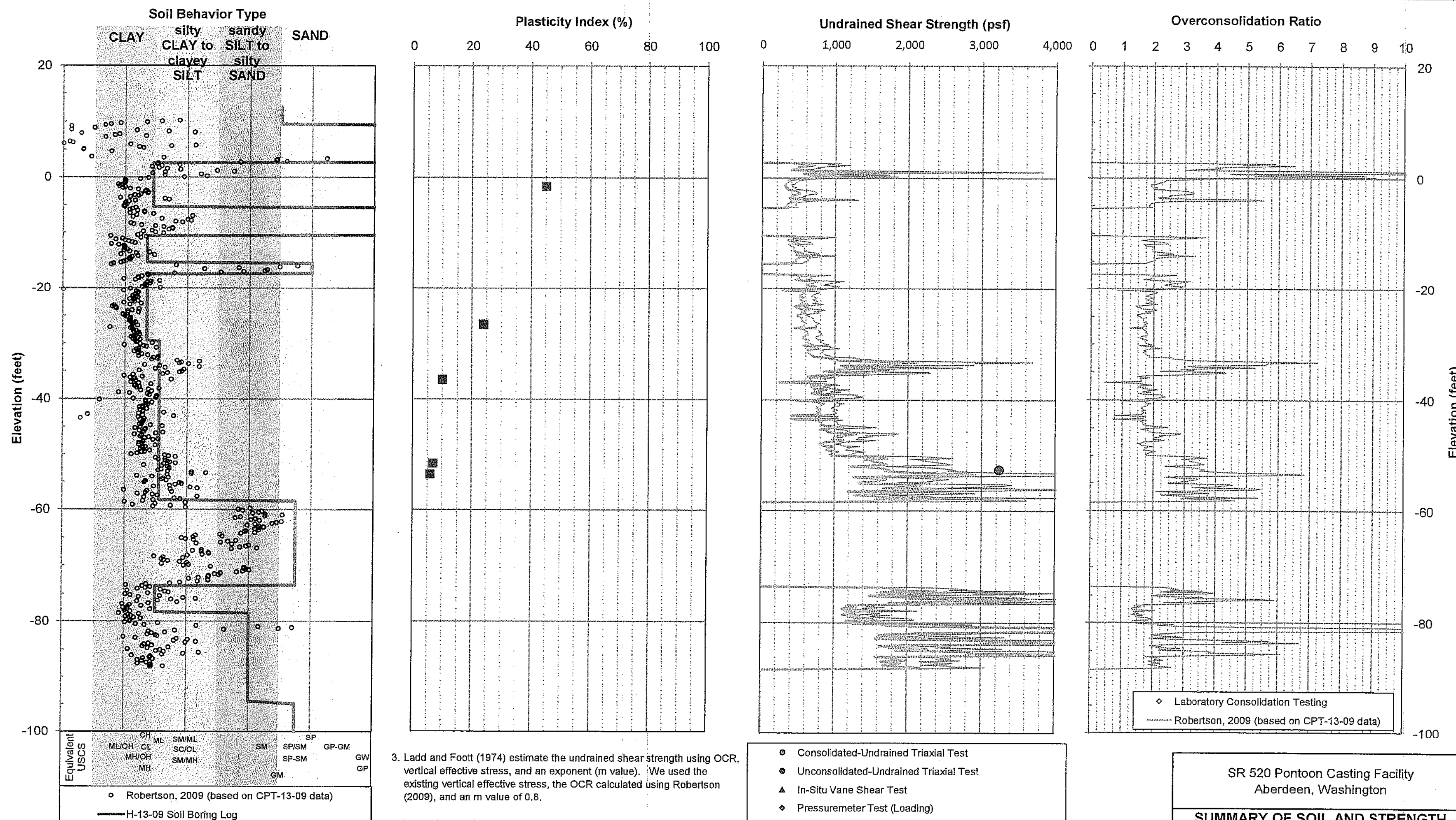
FIG. D-16

Summary of CPT Data.xlsm Printed: 2/15/2011 2:40 PM



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-13-09, which is in relatively close proximity to CPT-13-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-13-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-13-09.
5. CPT = cone penetration test
   OCR = overconsolidation ratio
   psf = pounds per square foot

- Consolidated-Undrained Triaxial Test
- Unconsolidated-Undrained Triaxial Test
- In-Situ Vane Shear Test
- Pressuremeter Test (Loading)
- Pressuremeter Test (Unloading)
- Static Direct Simple Shear Test
- Post-Cyclic Direct Simple Shear Test
- Robertson, 2009 (based on CPT-13-09 data)
- Ladd and Foott, 1974 (based on CPT OCR estimates)

SR 520 Pontoon Casting Facility
Aberdeen, Washington

SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-13-09 AND BORING H-13-09

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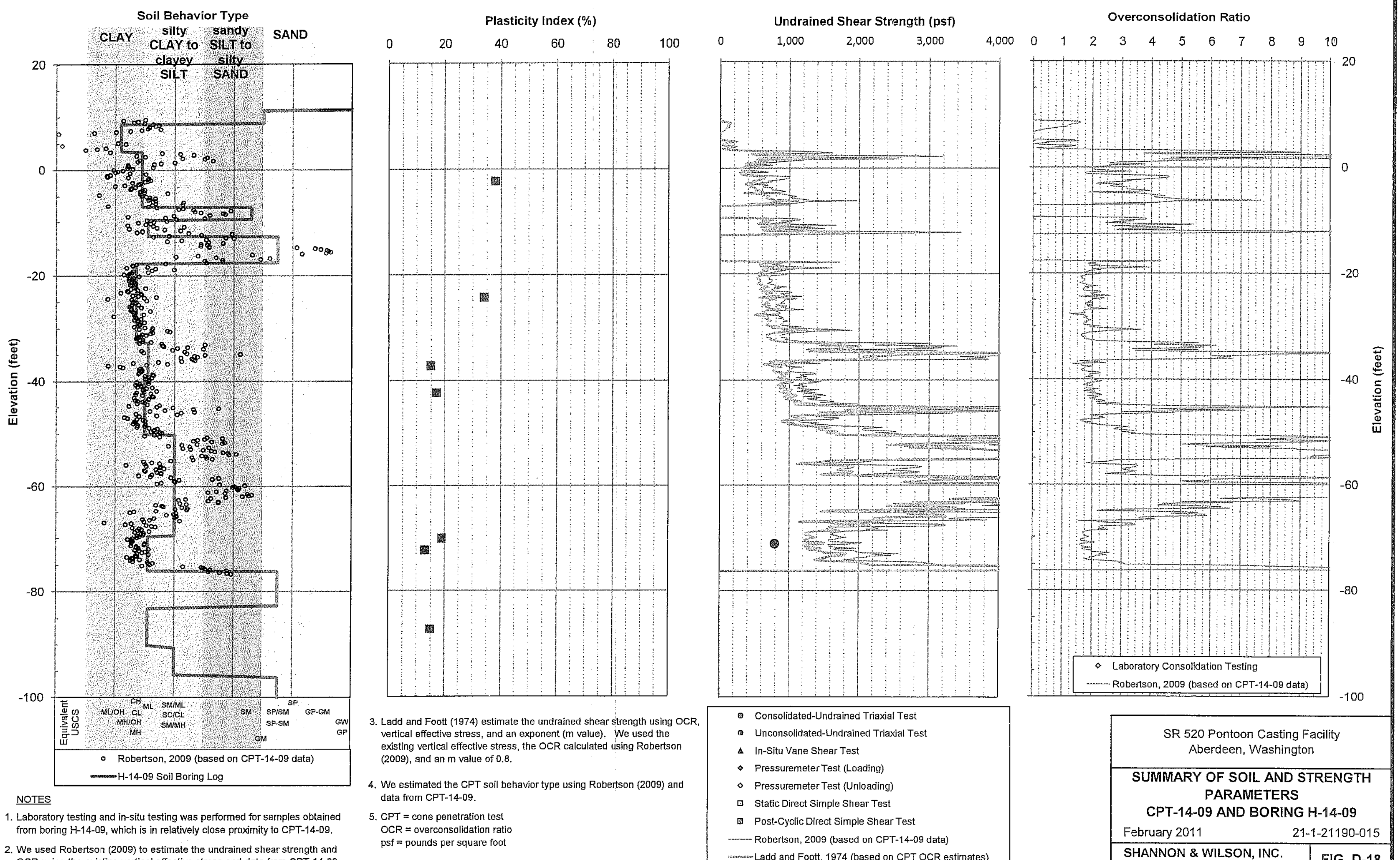
FIG. D-17

Summary of CPT Data.xlsm Printed: 2/15/2011 2:40 PM



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-14-09, which is in relatively close proximity to CPT-14-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-14-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-14-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

SR 520 Pontoon Casting Facility
Aberdeen, Washington

SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-14-09 AND BORING H-14-09

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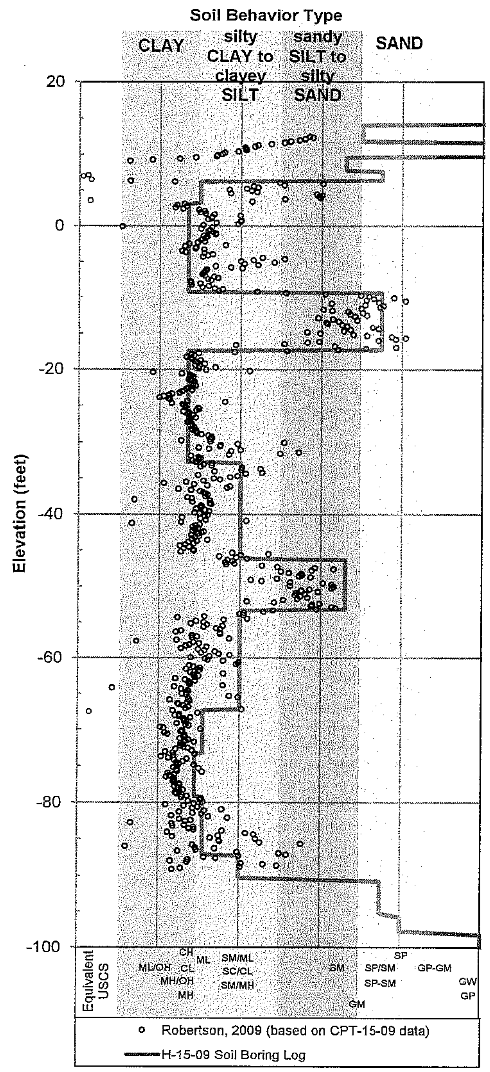
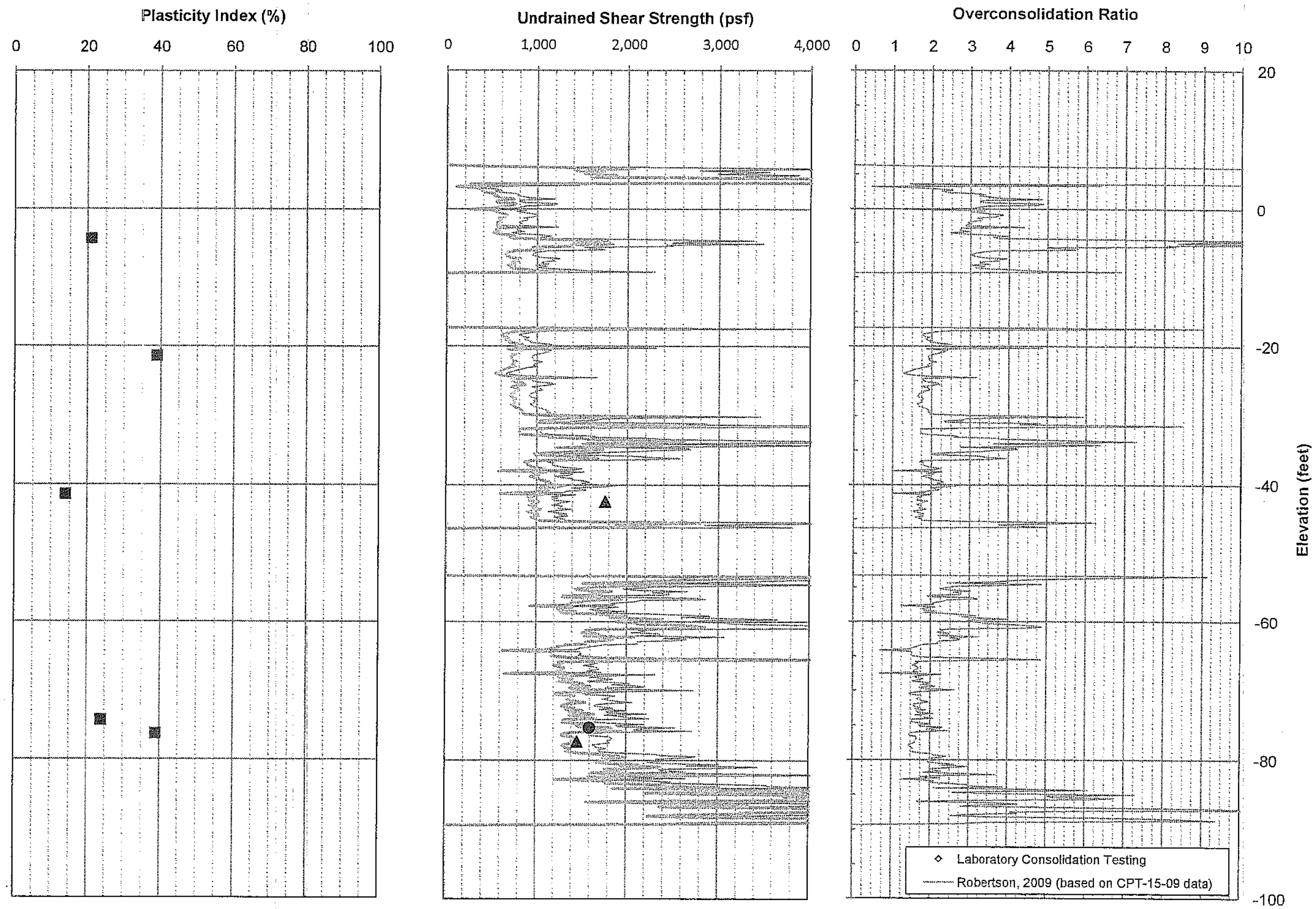
FIG. D-18

Summary of CPT Data.xlsm Printed: 2/15/2011 2:40 PM





Consolidated-Undrained Triaxial Test
Unconsolidated-Undrained Triaxial Test
In-Situ Vane Shear Test
Pressuremeter Test (Loading)
Pressuremeter Test (Unloading)
Static Direct Simple Shear Test
Post-Cyclic Direct Simple Shear Test
Robertson, 2009 (based on CPT-15-09 data)
Ladd and Foott, 1974 (based on CPT OCR estimates)

NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-15-09, which is in relatively close proximity to CPT-15-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-15-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-15-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

SR 520 Pontoon Casting Facility
Aberdeen, Washington

SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-15-09 AND BORING H-15-09

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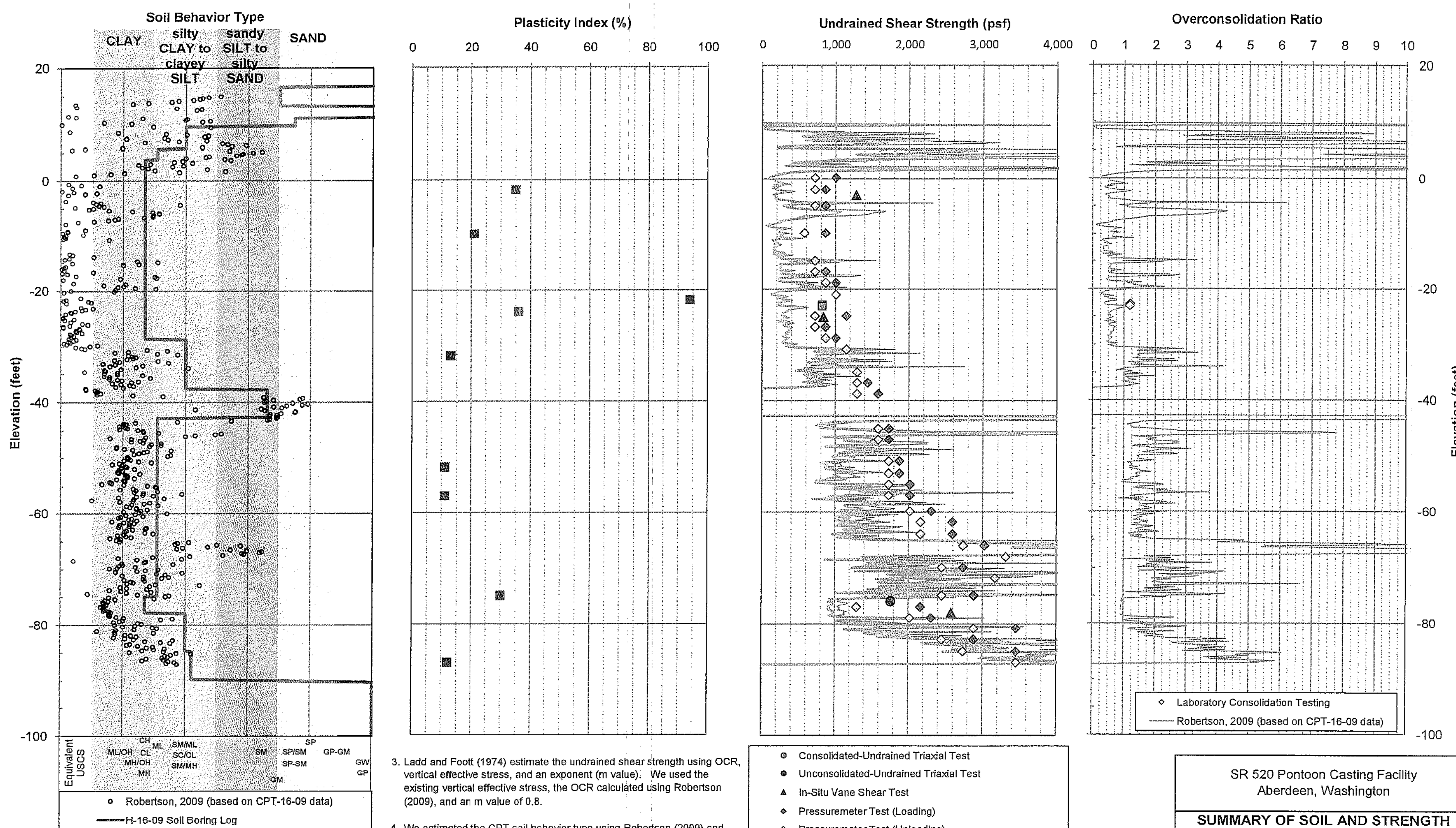
FIG. D-19

Summary of CPT Data.xlsm Printed: 2/15/2011 2:40 PM



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-16-09, which is in relatively close proximity to CPT-16-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-16-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-16-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-16-09 AND BORING H-16-09

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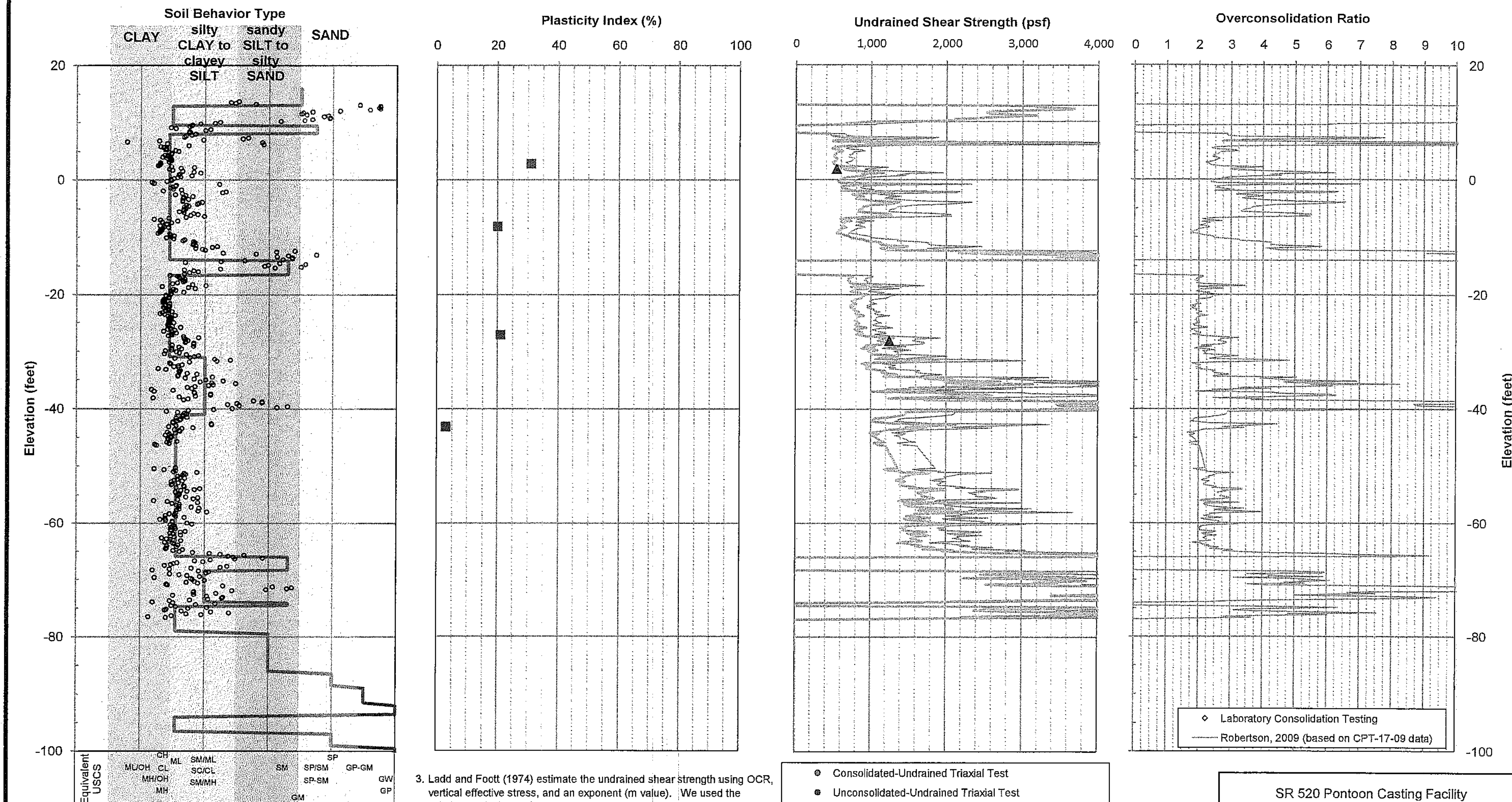
FIG. D-20



○ Robertson, 2009 (based on CPT-17-09 data)

H-17-09 Soil Boring Log

NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-17-09, which is in relatively close proximity to CPT-17-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-17-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-17-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

- Consolidated-Undrained Triaxial Test
- Unconsolidated-Undrained Triaxial Test
- ▲ In-Situ Vane Shear Test
- Pressuremeter Test (Loading)
- ◇ Pressuremeter Test (Unloading)
- □ Static Direct Simple Shear Test
- Post-Cyclic Direct Simple Shear Test
- Robertson, 2009 (based on CPT-17-09 data)
- Ladd and Foott, 1974 (based on CPT OCR estimates)

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-17-09 AND BORING H-17-09

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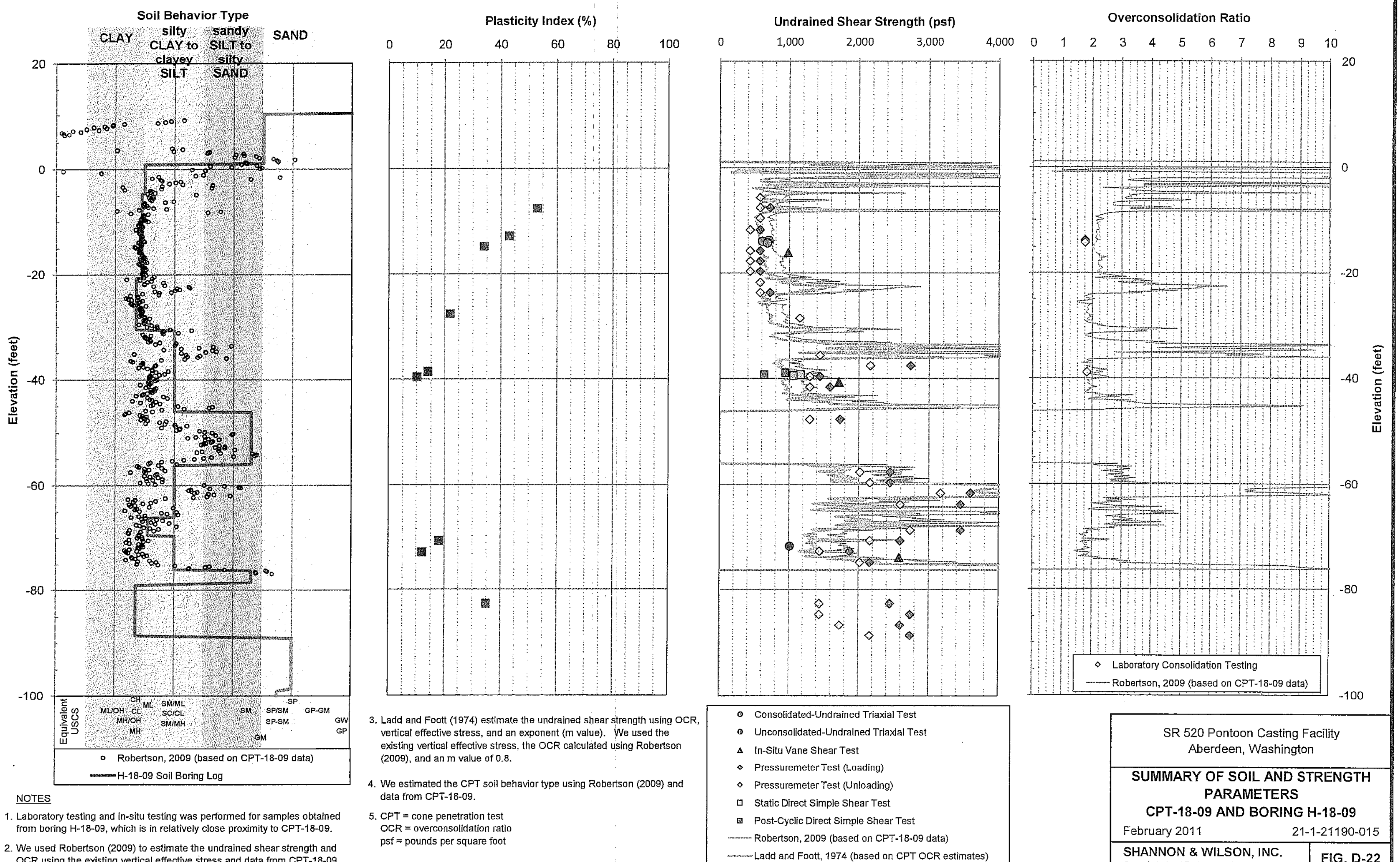
FIG. D-21



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-18-09, which is in relatively close proximity to CPT-18-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-18-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-18-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-18-09 AND BORING H-18-09

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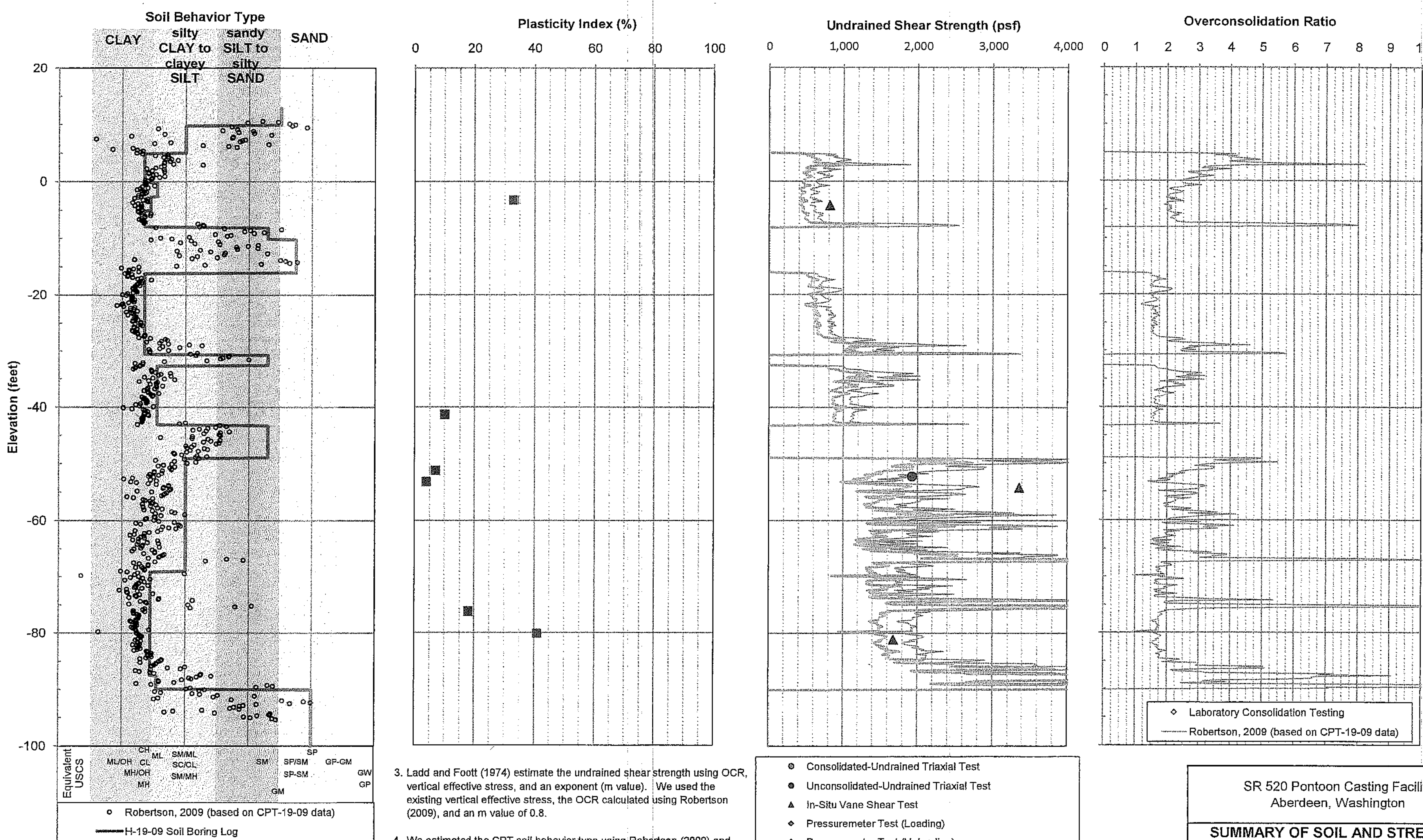
FIG. D-22



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-19-09, which is in relatively close proximity to CPT-19-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-19-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-19-09.
5. CPT = cone penetration test
   OCR = overconsolidation ratio
   psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-19-09 AND BORING H-19-09

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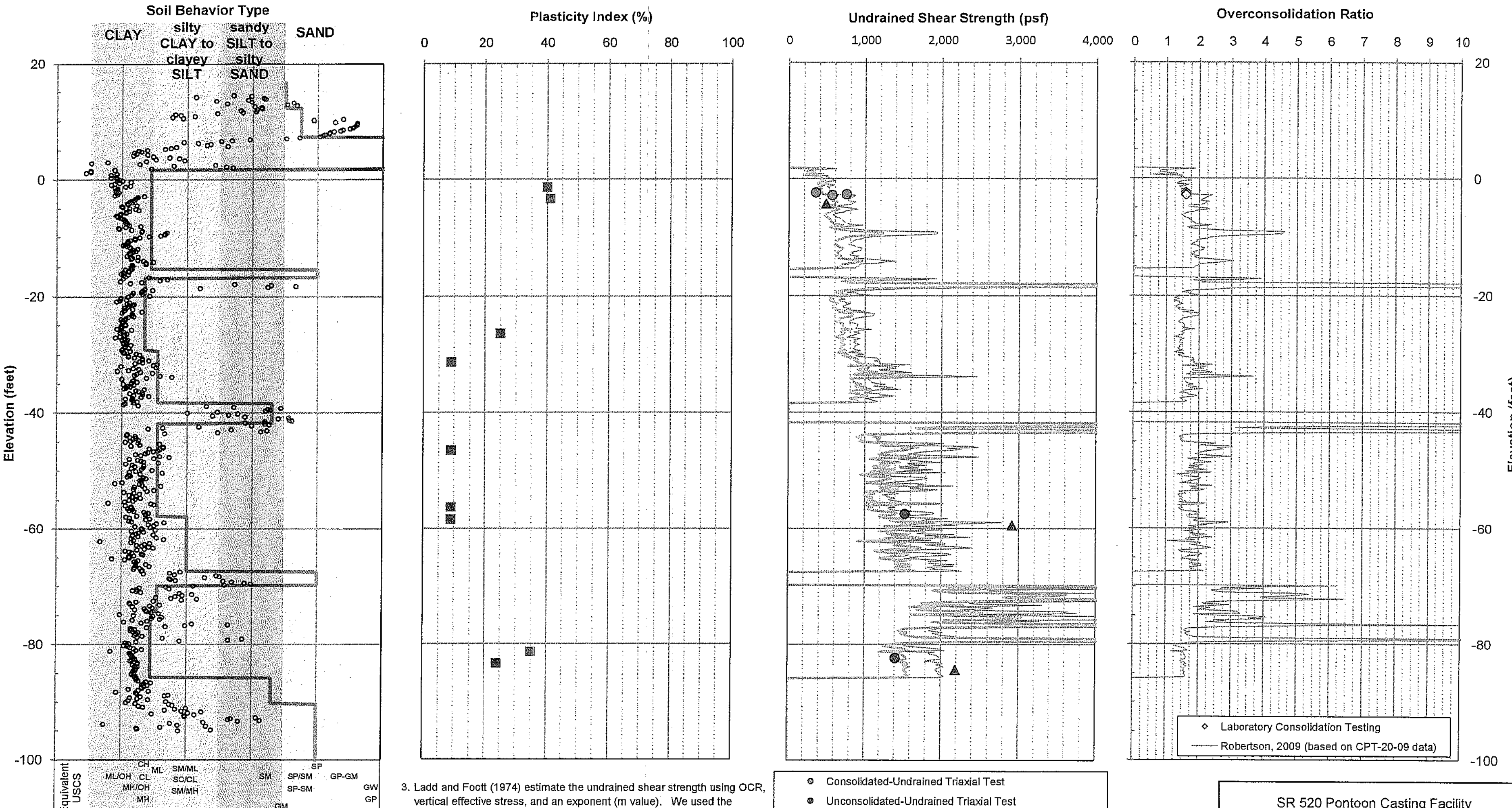
FIG. D-23

Summary of CPT Data.xlsm Printed: 2/15/2011 2:41 PM



- Robertson, 2009 (based on CPT-20-09 data)
- H-20-09 Soil Boring Log

NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-20-09, which is in relatively close proximity to CPT-20-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-20-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-20-09.
5. CPT = cone penetration test
   OCR = overconsolidation ratio
   psf = pounds per square foot

- Consolidated-Undrained Triaxial Test
- Unconsolidated-Undrained Triaxial Test
- In-Situ Vane Shear Test
- Pressuremeter Test (Loading)
- Pressuremeter Test (Unloading)
- Static Direct Simple Shear Test
- Post-Cyclic Direct Simple Shear Test
- Robertson, 2009 (based on CPT-20-09 data)
- Ladd and Foott, 1974 (based on CPT OCR estimates)

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-20-09 AND BORING H-20-09

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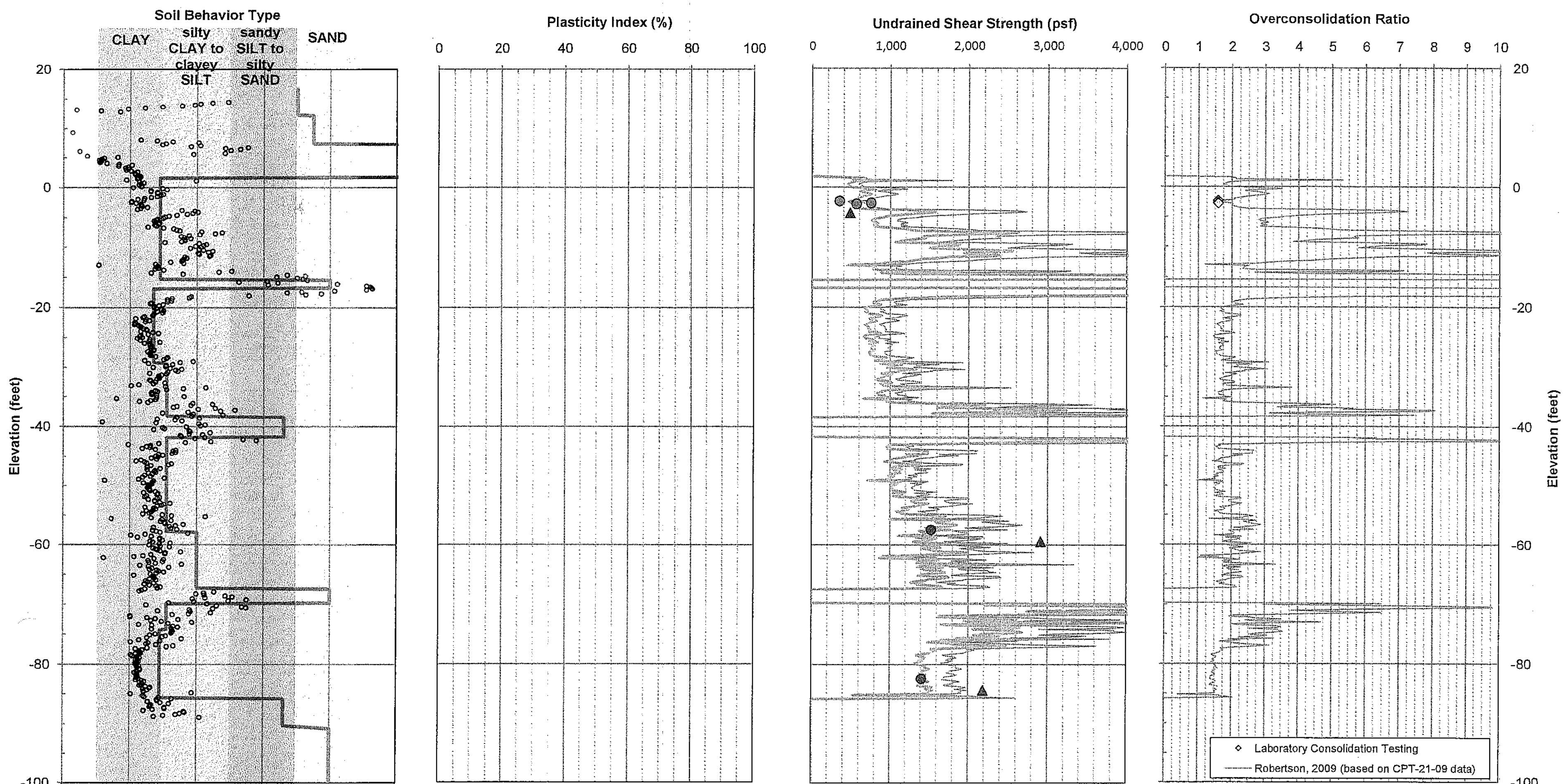
FIG. D-24



- Robertson, 2009 (based on CPT-21-09 data)
- H-20-09 Soil Boring Log

- Consolidated-Undrained Triaxial Test
- Unconsolidated-Undrained Triaxial Test
- In-Situ Vane Shear Test
- Pressuremeter Test (Loading)
- Pressuremeter Test (Unloading)
- Static Direct Simple Shear Test
- Post-Cyclic Direct Simple Shear Test
- Robertson, 2009 (based on CPT-21-09 data)
- Ladd and Foott, 1974 (based on CPT OCR estimates)

NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-20-09, which is in relatively close proximity to CPT-21-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-21-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-21-09.
5. CPT = cone penetration test
   OCR = overconsolidation ratio
   psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-21-09 AND BORING H-20-09

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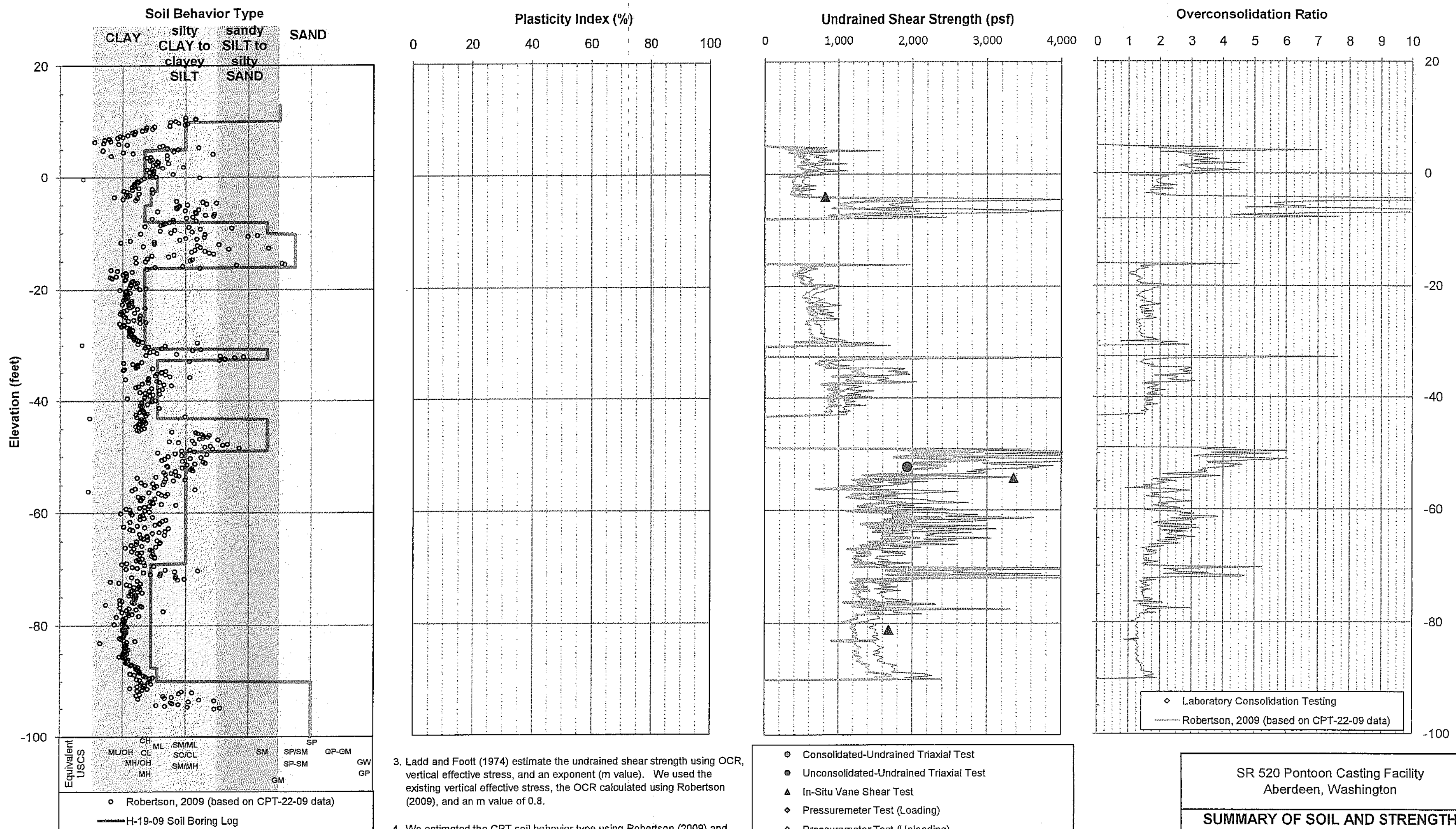
FIG. D-25



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-19-09, which is in relatively close proximity to CPT-22-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-22-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-22-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

- Consolidated-Undrained Triaxial Test
- Unconsolidated-Undrained Triaxial Test
- In-Situ Vane Shear Test
- Pressuremeter Test (Loading)
- Pressuremeter Test (Unloading)
- Static Direct Simple Shear Test
- Post-Cyclic Direct Simple Shear Test
- Robertson, 2009 (based on CPT-22-09 data)
- Ladd and Foott, 1974 (based on CPT OCR estimates)

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-22-09 AND BORING H-19-09

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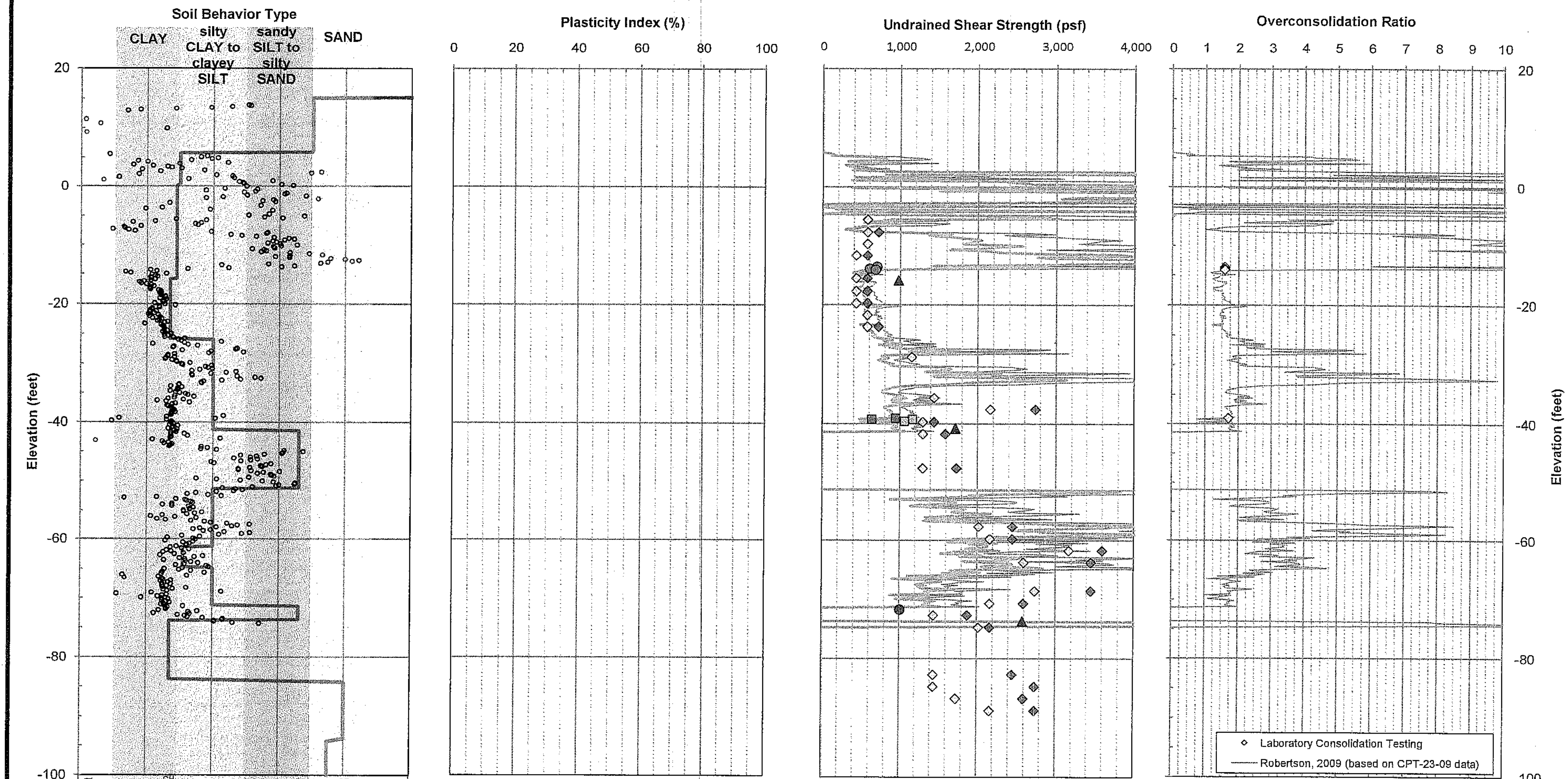
FIG. D-26



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-18-09, which is in relatively close proximity to CPT-23-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-23-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-23-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-23-09 AND BORING H-18-09

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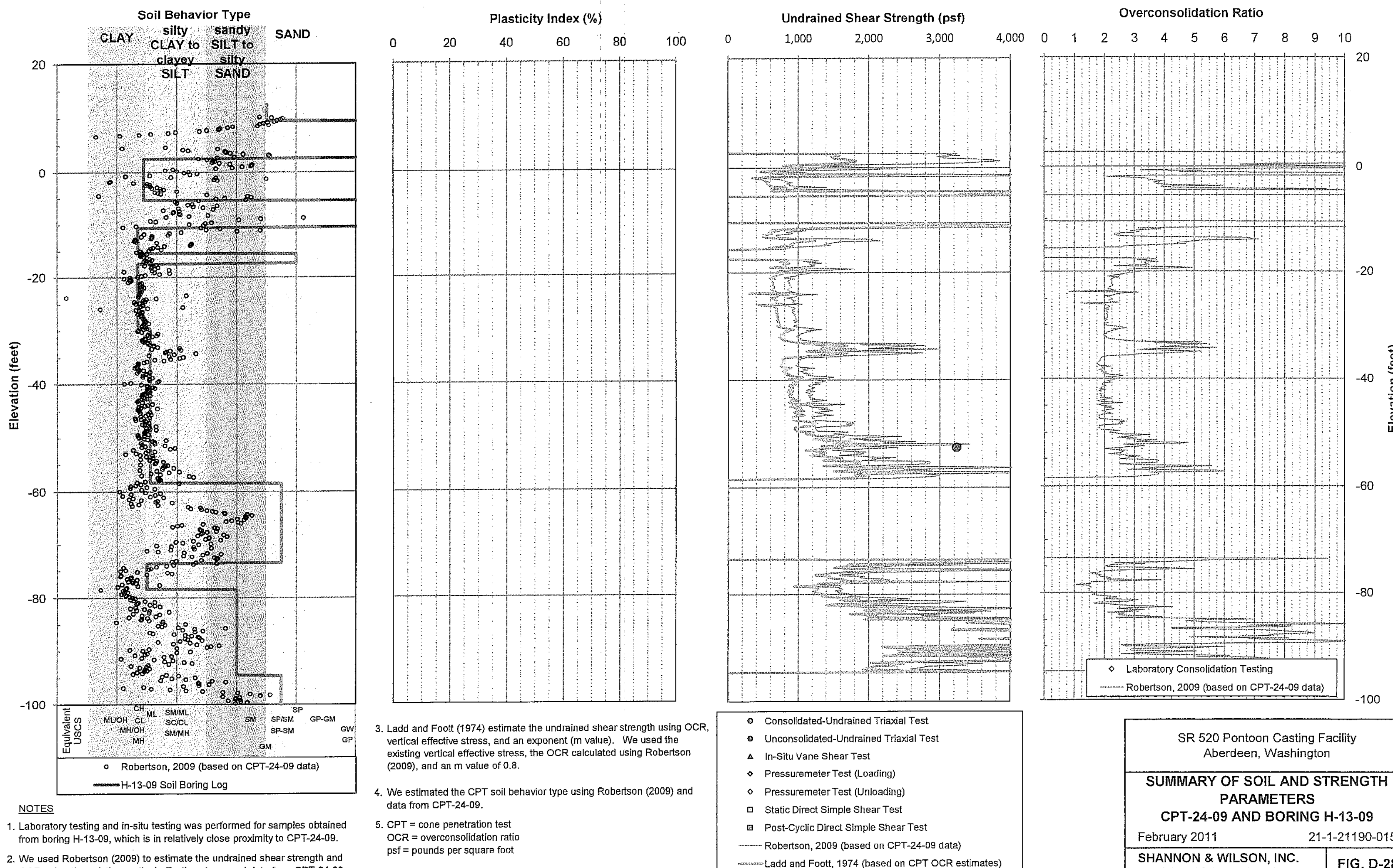
FIG. D-27

Summary of CPT Data.xlsm Printed: 2/15/2011 2:41 PM



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-13-09, which is in relatively close proximity to CPT-24-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-24-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-24-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-24-09 AND BORING H-13-09

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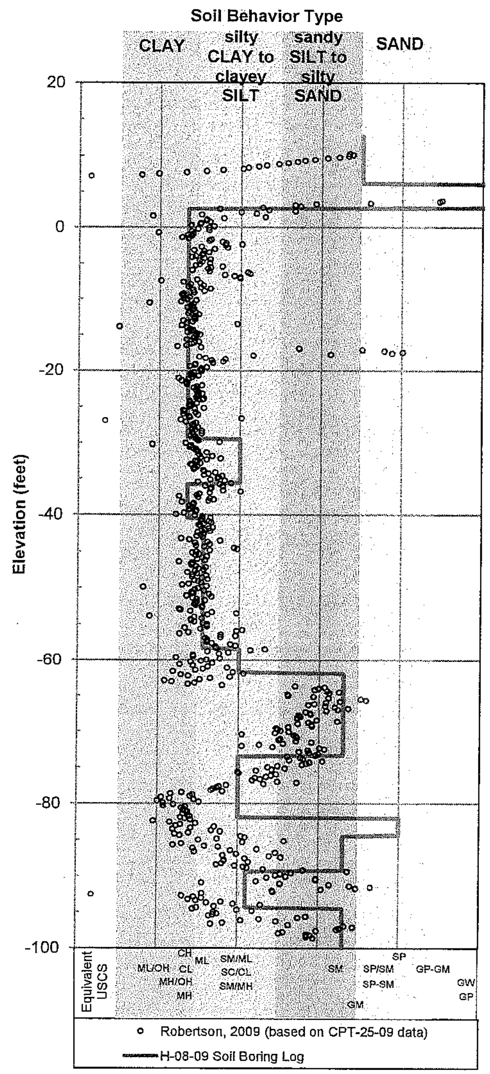
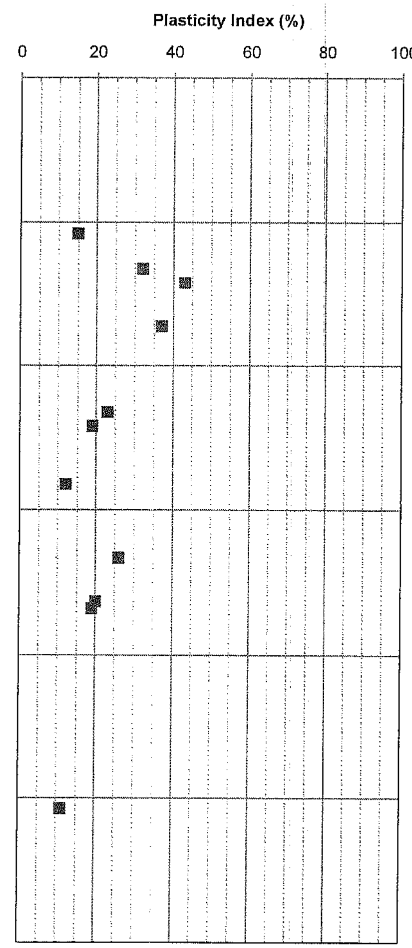
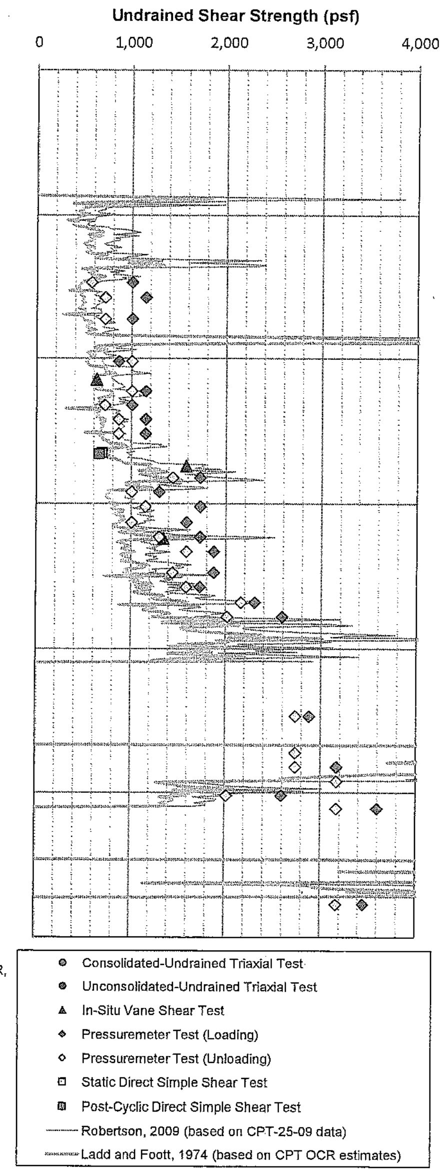
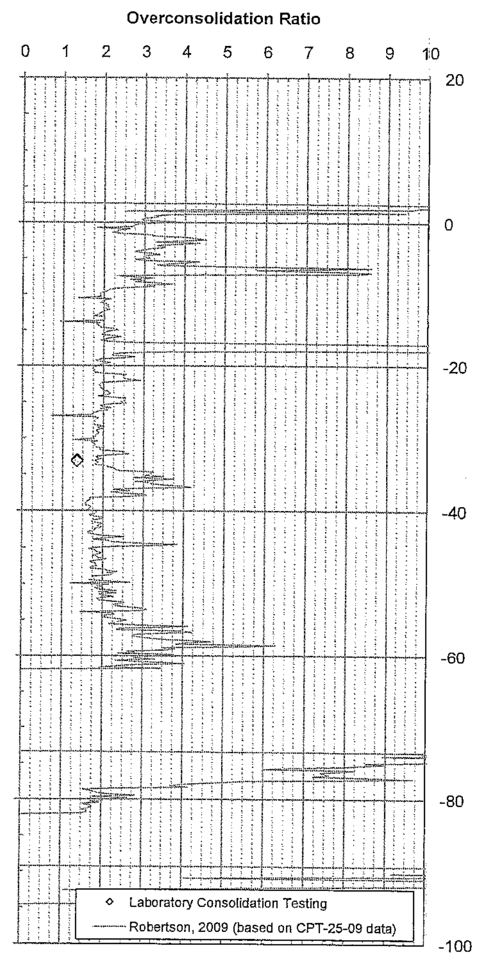
FIG. D-28

Summary of CPT Data.xlsm Printed: 2/15/2011 2:41 PM









NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-08-09, which is in relatively close proximity to CPT-25-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-25-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-25-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-25-09 AND BORING H-08-09

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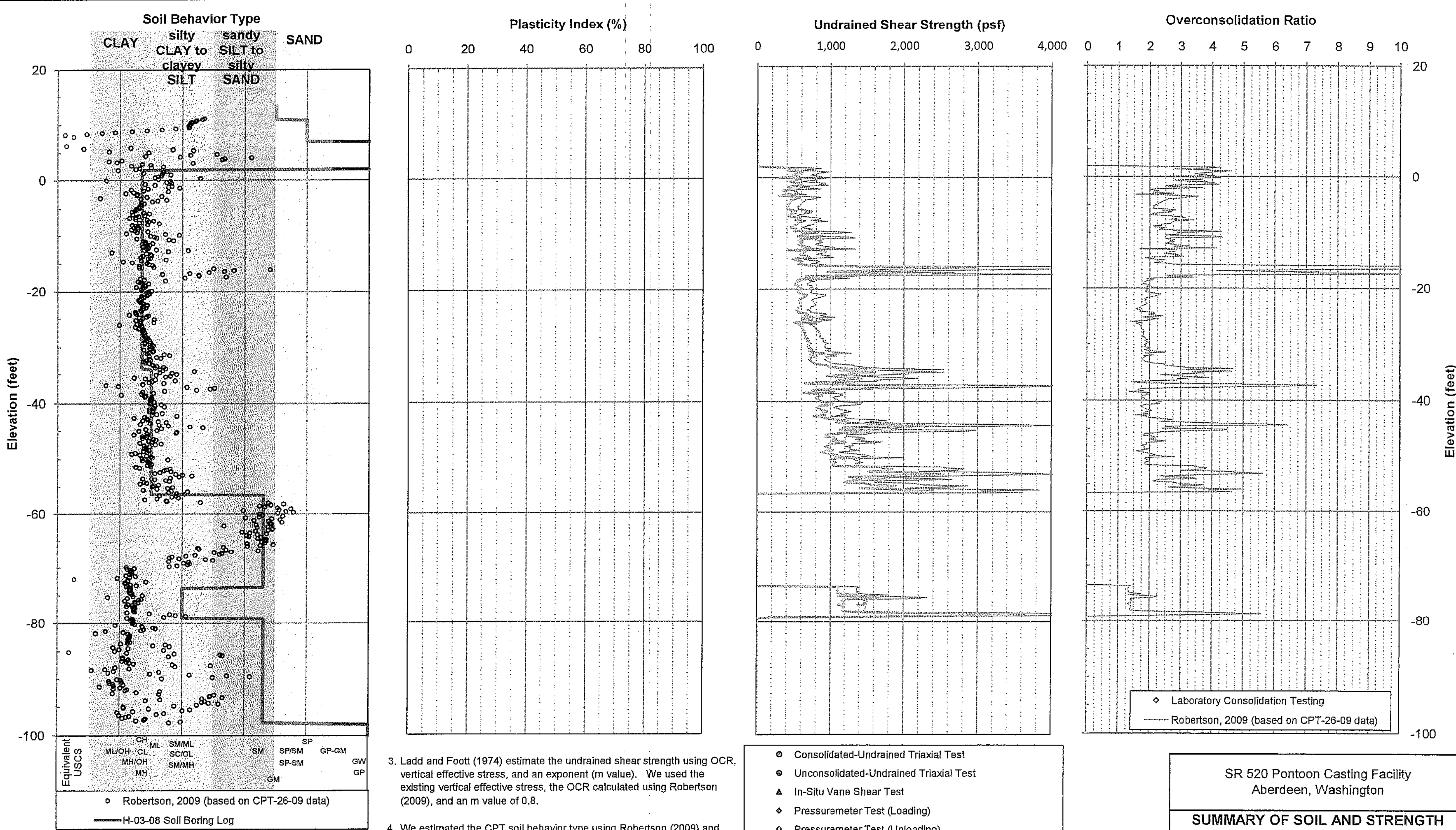
FIG. D-29

Summary of CPT Data.xlsm Printed: 2/15/2011 2:41 PM



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-03-08, which is in relatively close proximity to CPT-26-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-26-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-26-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-26-09 AND BORING H-03-08

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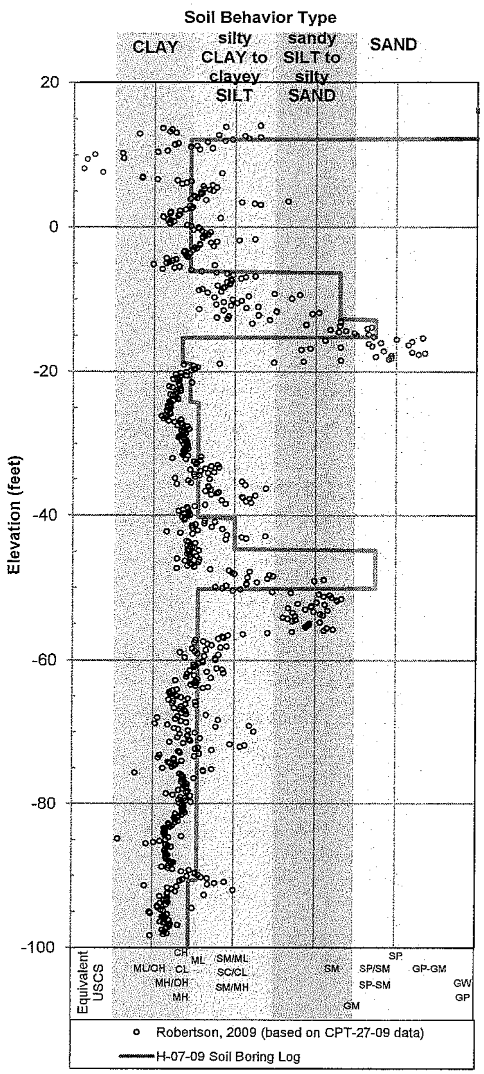
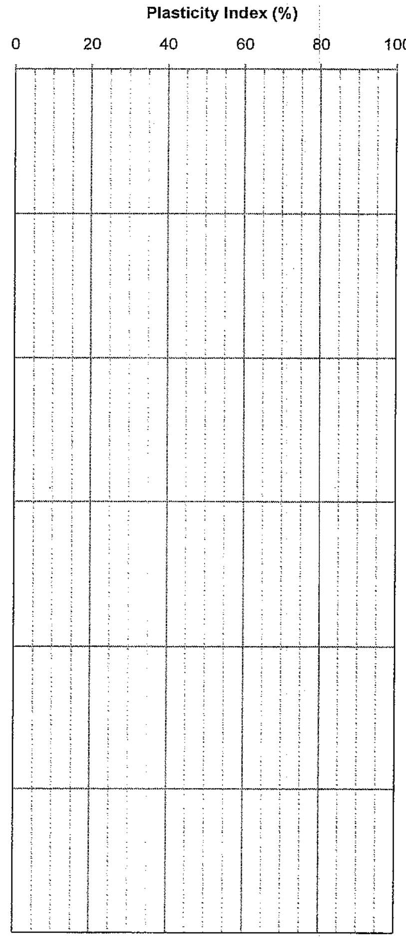
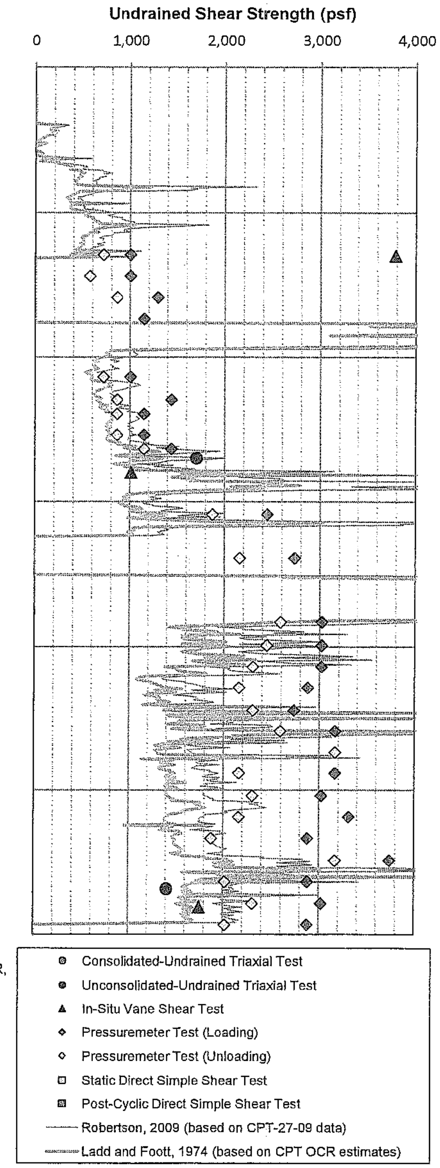
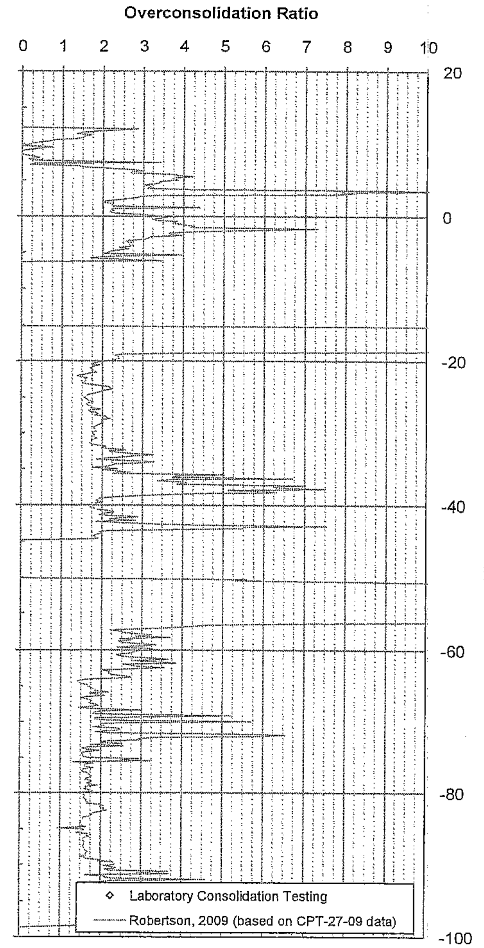
FIG. D-30

Summary of CPT Data.xlsm Printed: 2/15/2011 2:41 PM









NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-07-09, which is in relatively close proximity to CPT-27-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-27-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-27-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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CPT-27-09 AND BORING H-07-09

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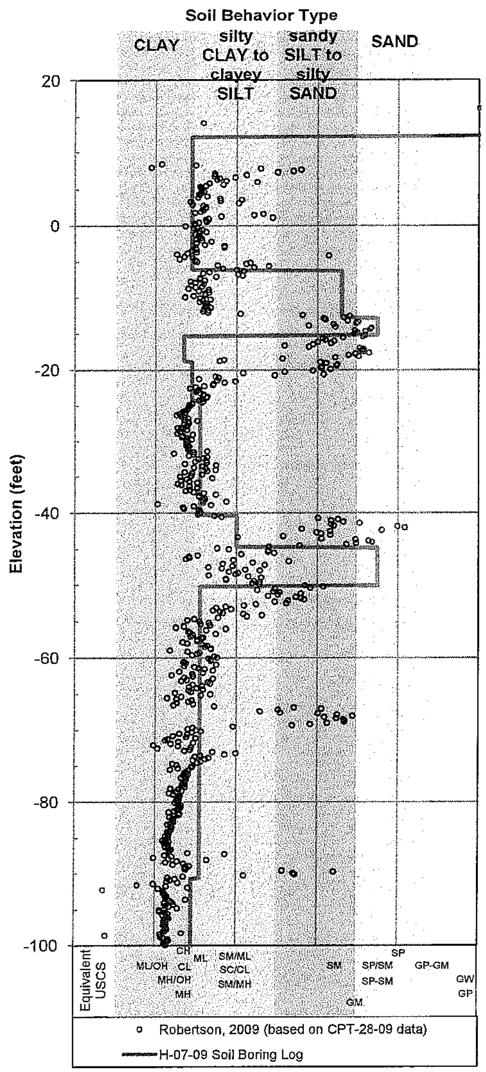
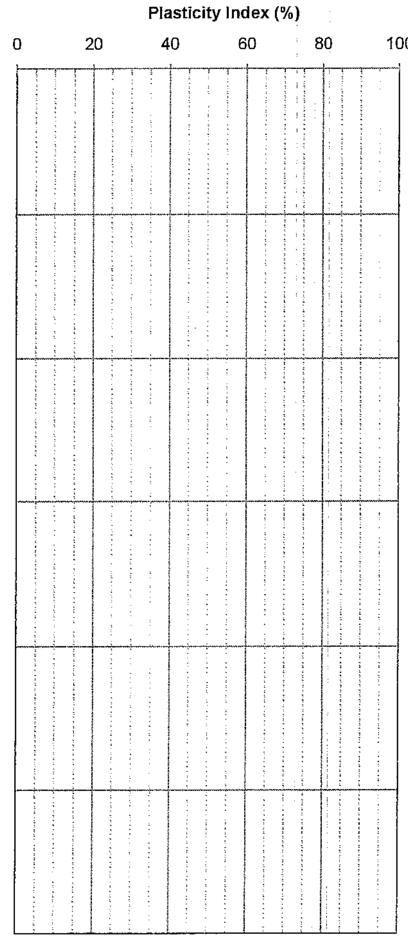
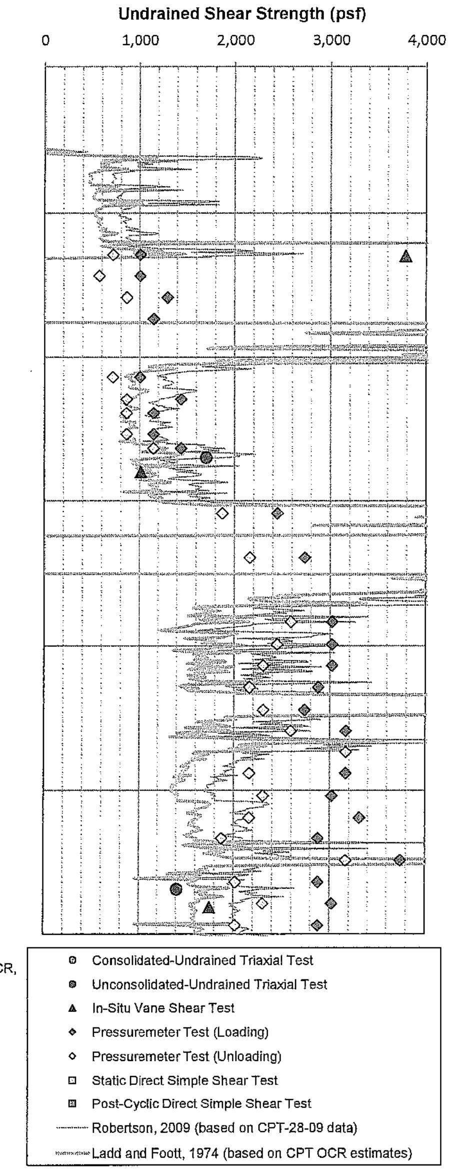
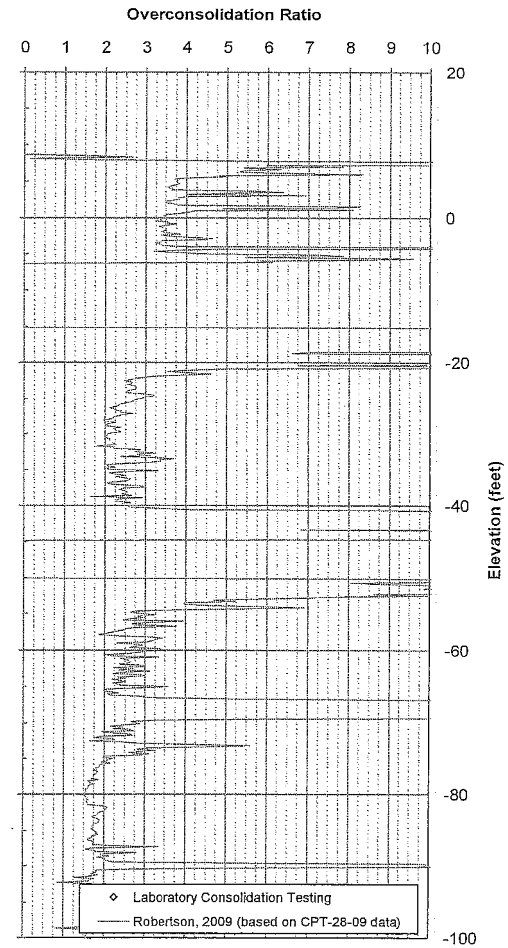
FIG. D-31

Summary of CPT Data.xlsm Printed: 2/15/2011 2:41 PM









NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-07-09, which is in relatively close proximity to CPT-28-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-28-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-28-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-28-09 AND BORING H-07-09

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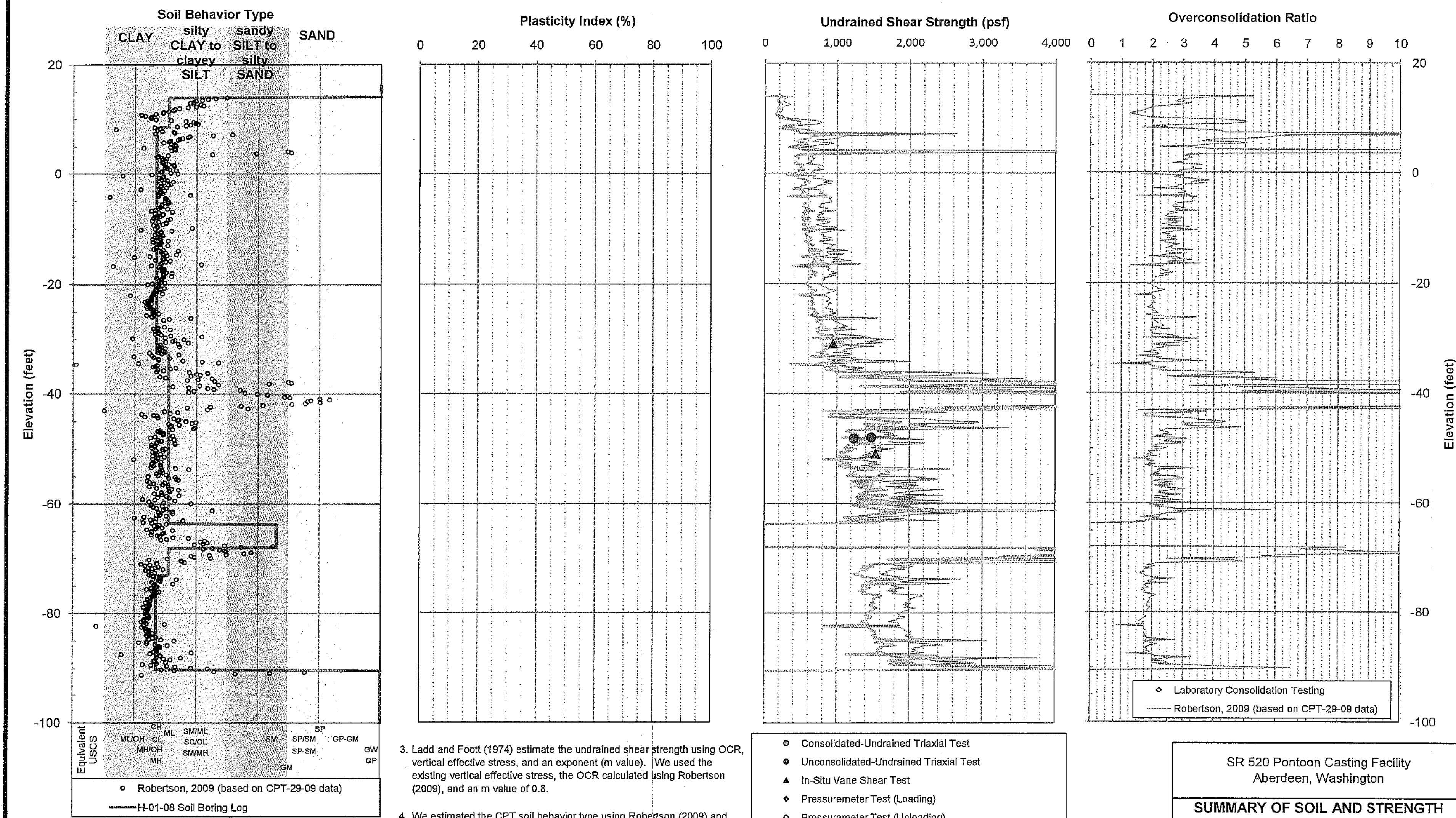
FIG. D-32



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-01-08, which is in relatively close proximity to CPT-29-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-29-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-29-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-29-09 AND BORING H-01-08

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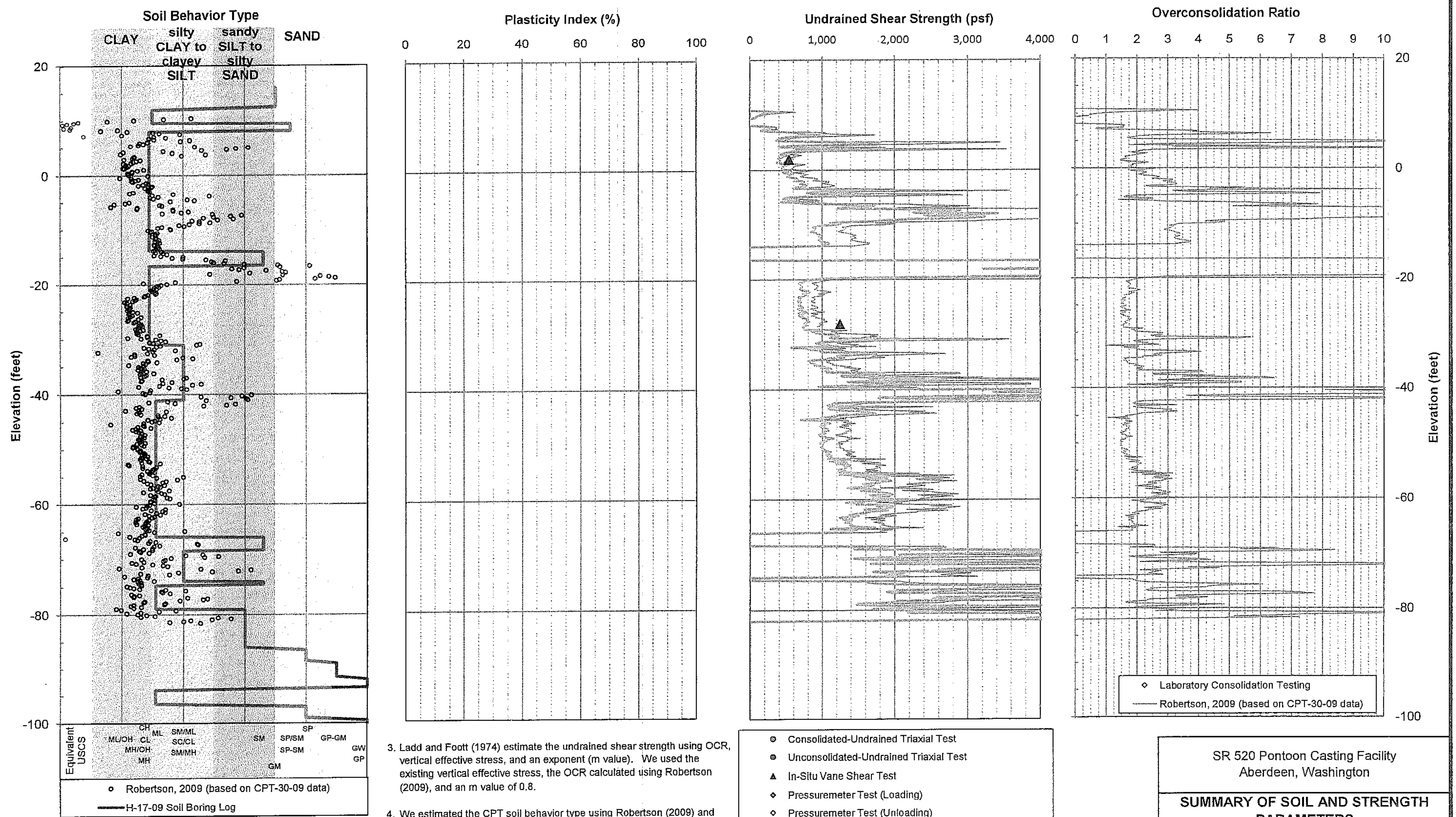
FIG. D-33



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-17-09, which is in relatively close proximity to CPT-30-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-30-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-30-09.
5. CPT = cone penetration test
   OCR = overconsolidation ratio
   psf = pounds per square foot

- Consolidated-Undrained Triaxial Test
- Unconsolidated-Undrained Triaxial Test
- In-Situ Vane Shear Test
- Pressuremeter Test (Loading)
- Pressuremeter Test (Unloading)
- Static Direct Simple Shear Test
- Post-Cyclic Direct Simple Shear Test
- Robertson, 2009 (based on CPT-30-09 data)
- Ladd and Foott, 1974 (based on CPT OCR estimates)

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CPT-30-09 AND BORING H-17-09

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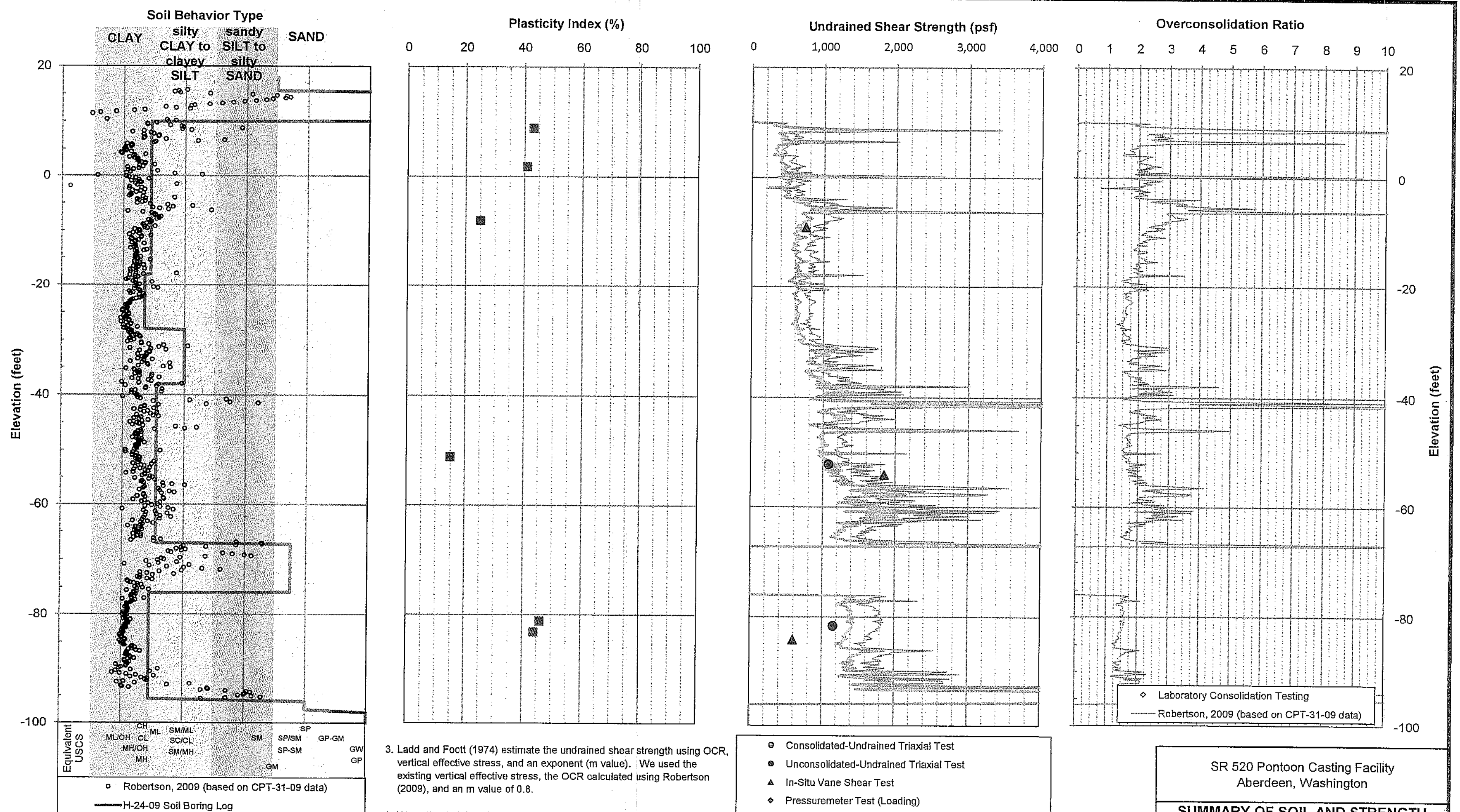
FIG. D-34



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-24-09, which is in relatively close proximity to CPT-31-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-31-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-31-09.
5. CPT = cone penetration test
   OCR = overconsolidation ratio
   psf = pounds per square foot

- Consolidated-Undrained Triaxial Test
- Unconsolidated-Undrained Triaxial Test
- In-Situ Vane Shear Test
- Pressuremeter Test (Loading)
- Pressuremeter Test (Unloading)
- Static Direct Simple Shear Test
- Post-Cyclic Direct Simple Shear Test
- Robertson, 2009 (based on CPT-31-09 data)
- Ladd and Foott, 1974 (based on CPT OCR estimates)

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CPT-31-09 AND BORING H-24-09

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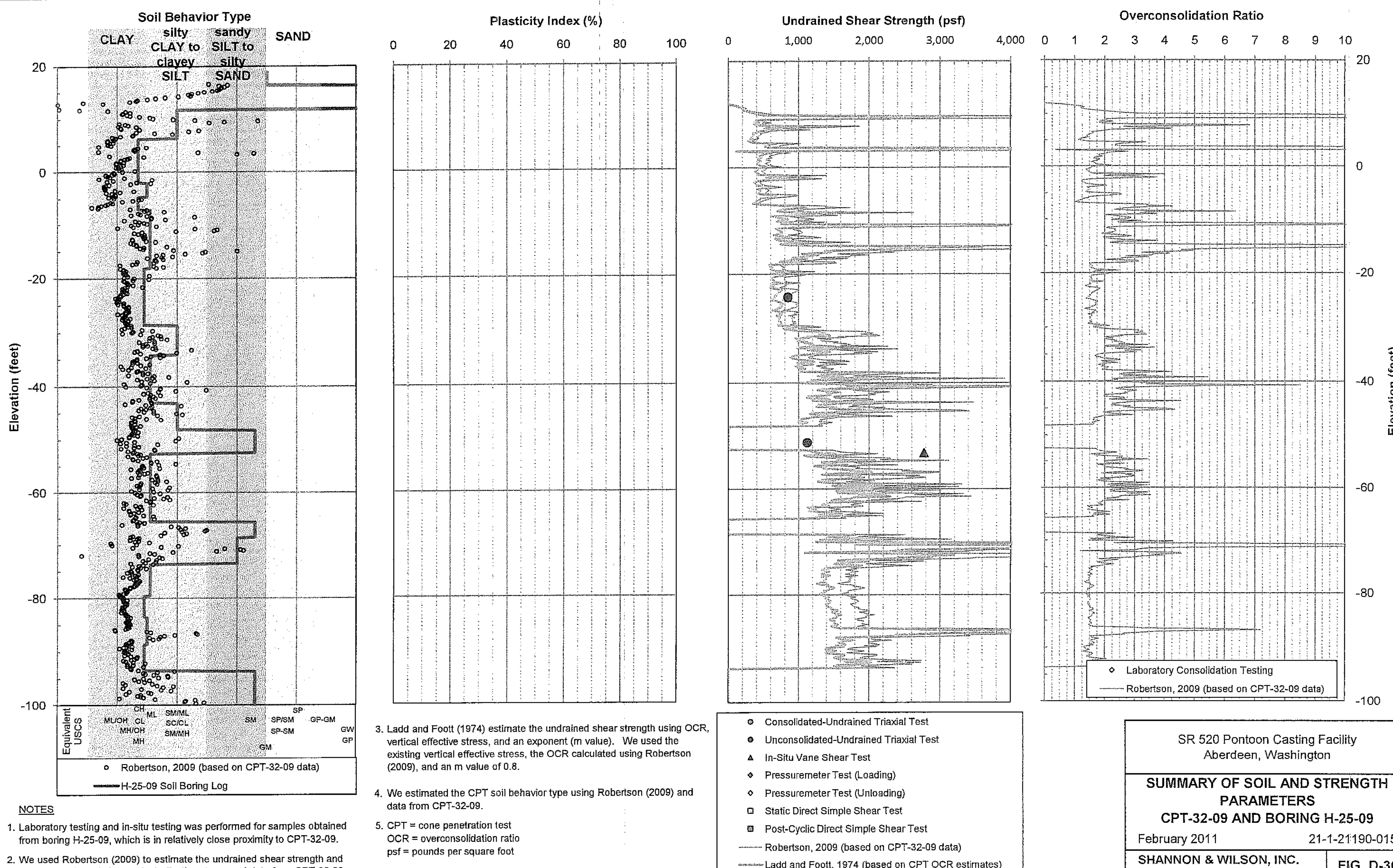
FIG. D-35



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-25-09, which is in relatively close proximity to CPT-32-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-32-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-32-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-32-09 AND BORING H-25-09

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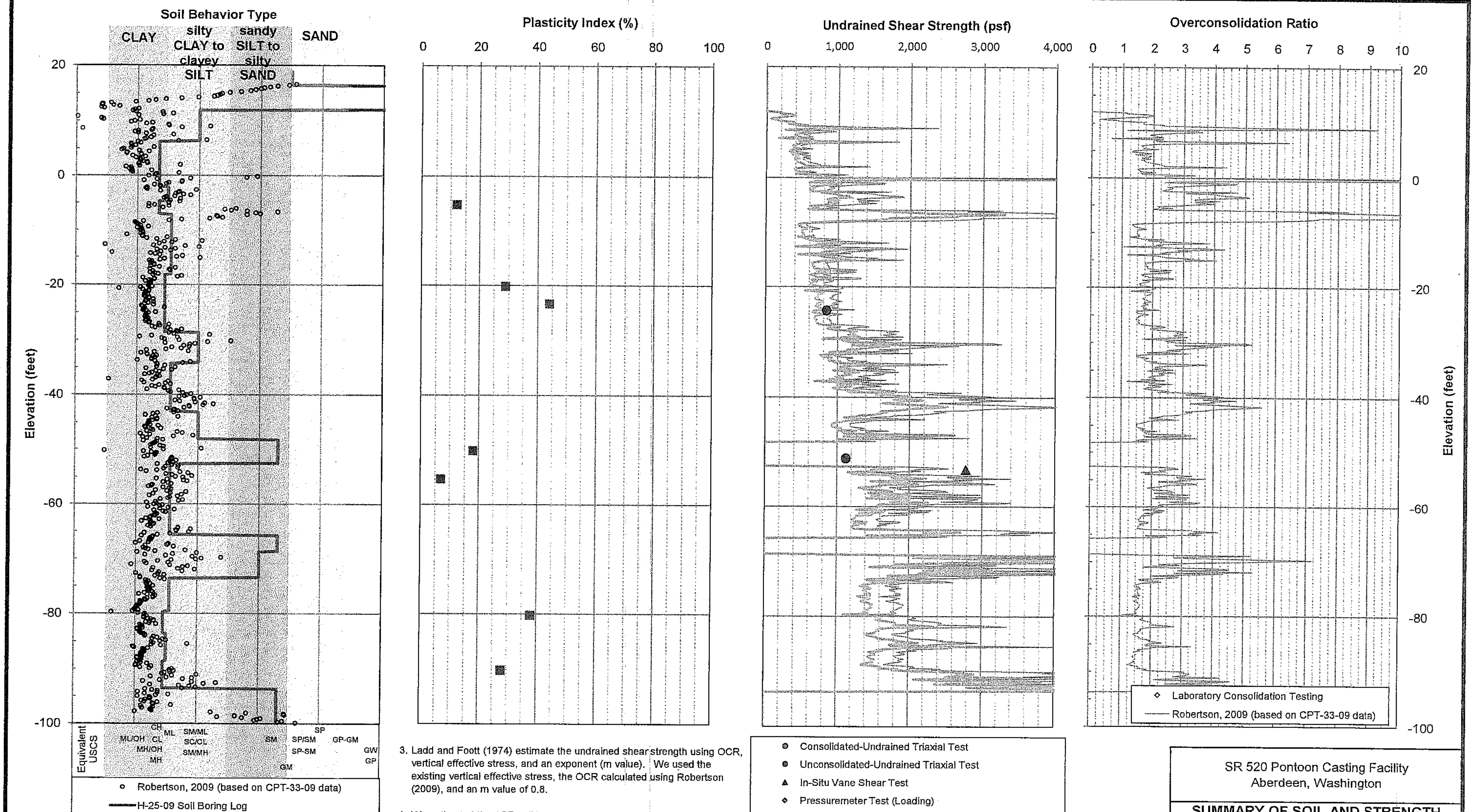
FIG. D-36



Consolidated-Undrained Triaxial Test
Unconsolidated-Undrained Triaxial Test
In-Situ Vane Shear Test
Pressuremeter Test (Loading)
Pressuremeter Test (Unloading)
Static Direct Simple Shear Test
Post-Cyclic Direct Simple Shear Test
Robertson, 2009 (based on CPT-33-09 data)
Ladd and Foott, 1974 (based on CPT OCR estimates)

NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-25-09, which is in relatively close proximity to CPT-33-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-33-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-33-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

SR 520 Pontoon Casting Facility
Aberdeen, Washington

SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-33-09 AND BORING H-25-09

February 2011 | 21-1-21190-015

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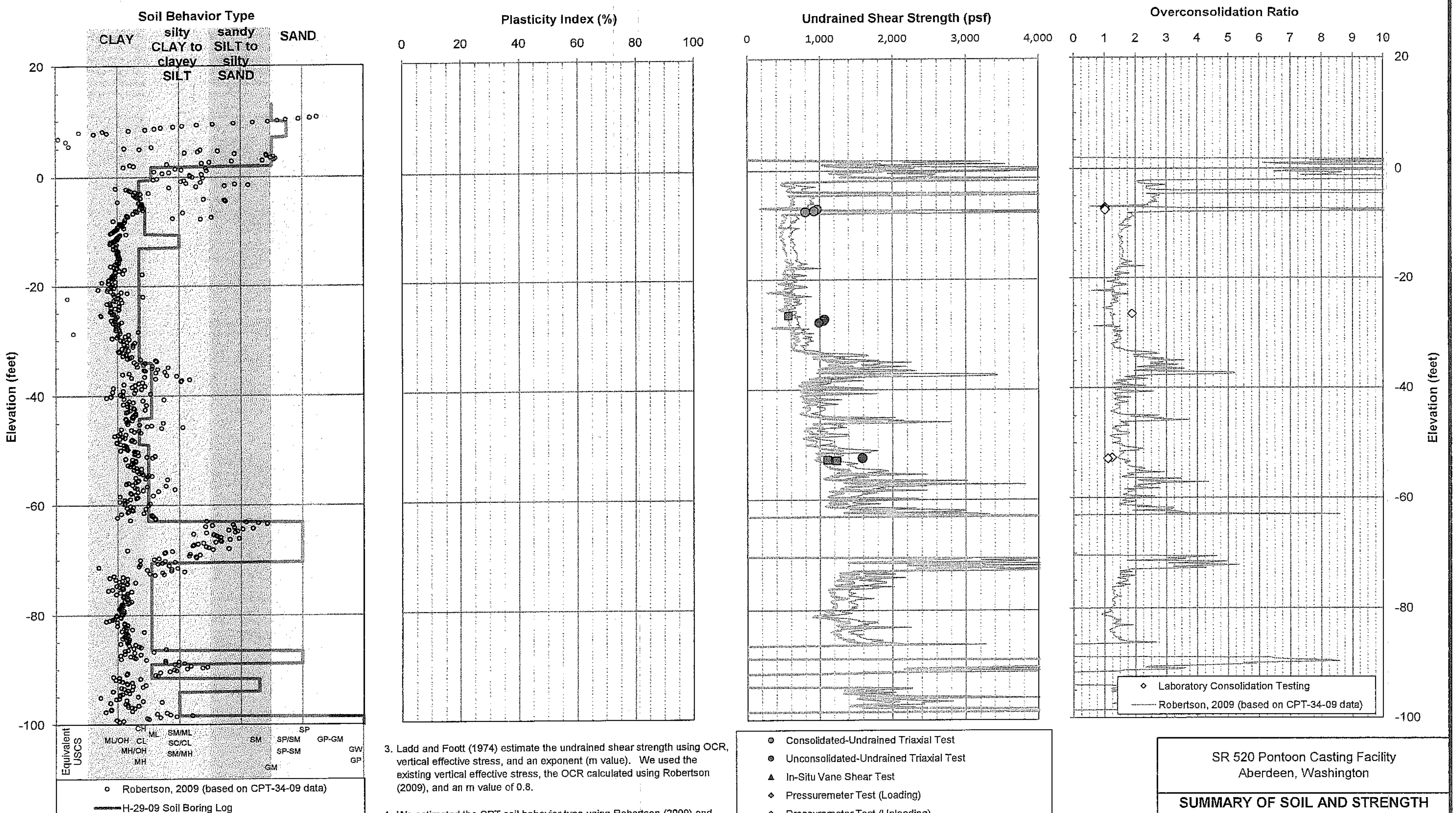
FIG. D-37

Summary of CPT Data.xlsm Printed: 2/15/2011 2:42 PM



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-29-09, which is in relatively close proximity to CPT-34-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-34-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-34-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-34-09 AND BORING H-29-09

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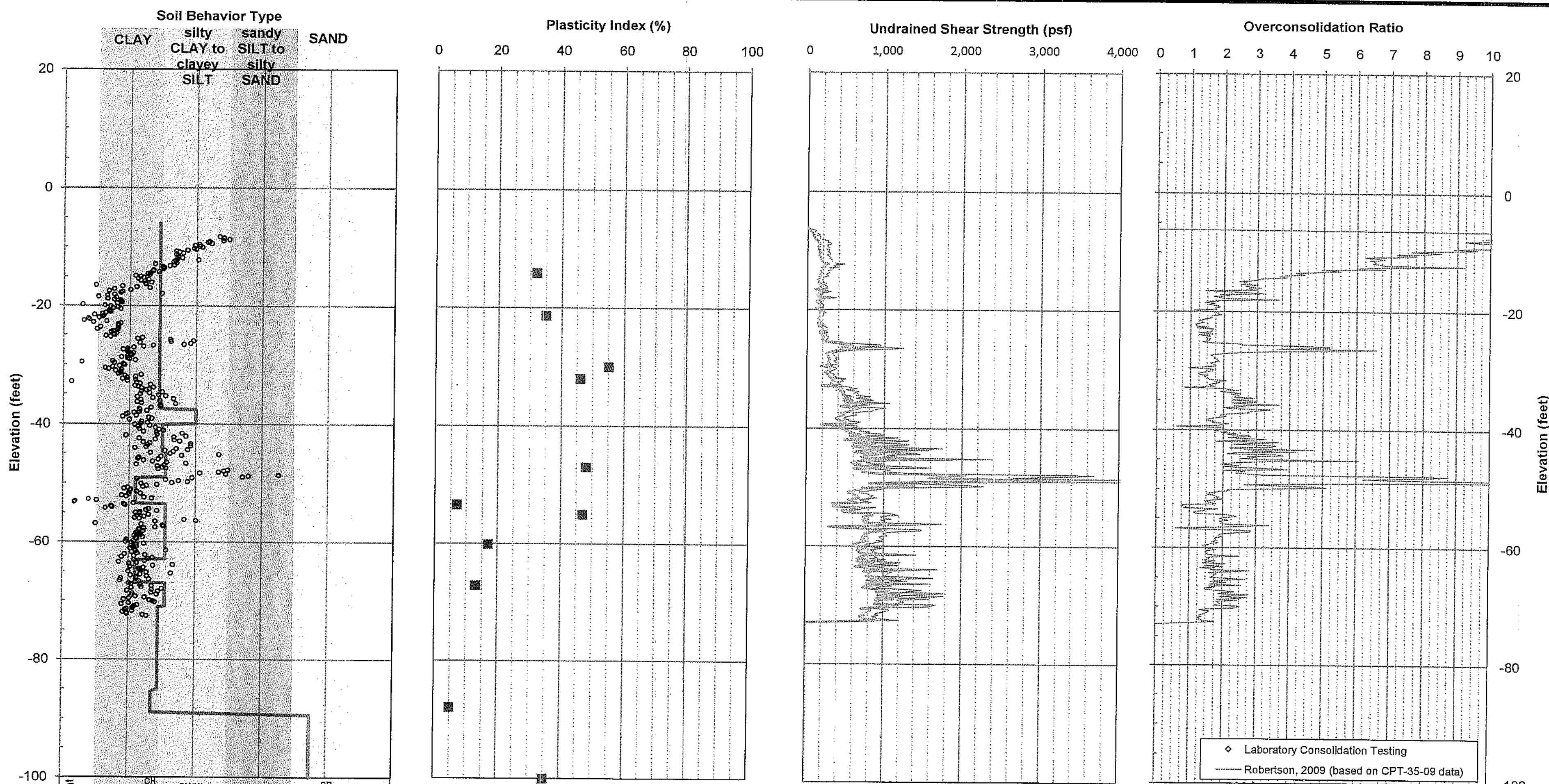
FIG. D-38



o Robertson, 2009 (based on CPT-35-09 data)

H-23-09 Soil Boring Log

NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-23-09, which is in relatively close proximity to CPT-35-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-35-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-35-09.
5. CPT = cone penetration test
   OCR = overconsolidation ratio
   psf = pounds per square foot

- Consolidated-Undrained Triaxial Test
- Unconsolidated-Undrained Triaxial Test
- In-Situ Vane Shear Test
- Pressuremeter Test (Loading)
- Pressuremeter Test (Unloading)
- Static Direct Simple Shear Test
- Post-Cyclic Direct Simple Shear Test
- Robertson, 2009 (based on CPT-35-09 data)
- Ladd and Foott, 1974 (based on CPT OCR estimates)

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-35-09 AND BORING H-23-09

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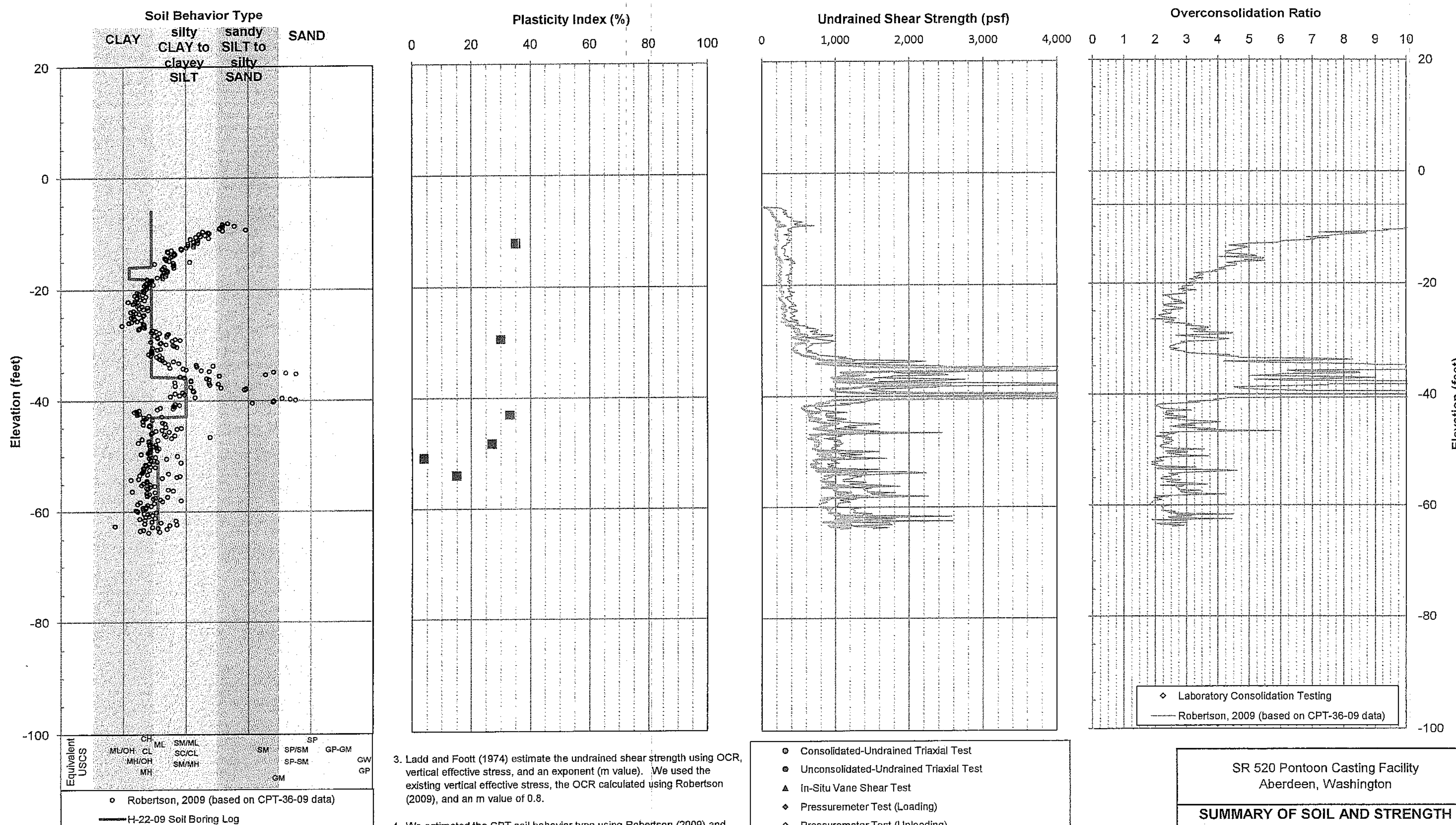
FIG. D-39

Summary of CPT Data.xlsm Printed: 2/15/2011 2:42 PM



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-22-09, which is in relatively close proximity to CPT-36-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-36-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-36-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

- Consolidated-Undrained Triaxial Test
- Unconsolidated-Undrained Triaxial Test
- In-Situ Vane Shear Test
- Pressuremeter Test (Loading)
- Pressuremeter Test (Unloading)
- Static Direct Simple Shear Test
- Post-Cyclic Direct Simple Shear Test
- Robertson, 2009 (based on CPT-36-09 data)
- Ladd and Foott, 1974 (based on CPT OCR estimates)

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SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-36-09 AND BORING H-22-09

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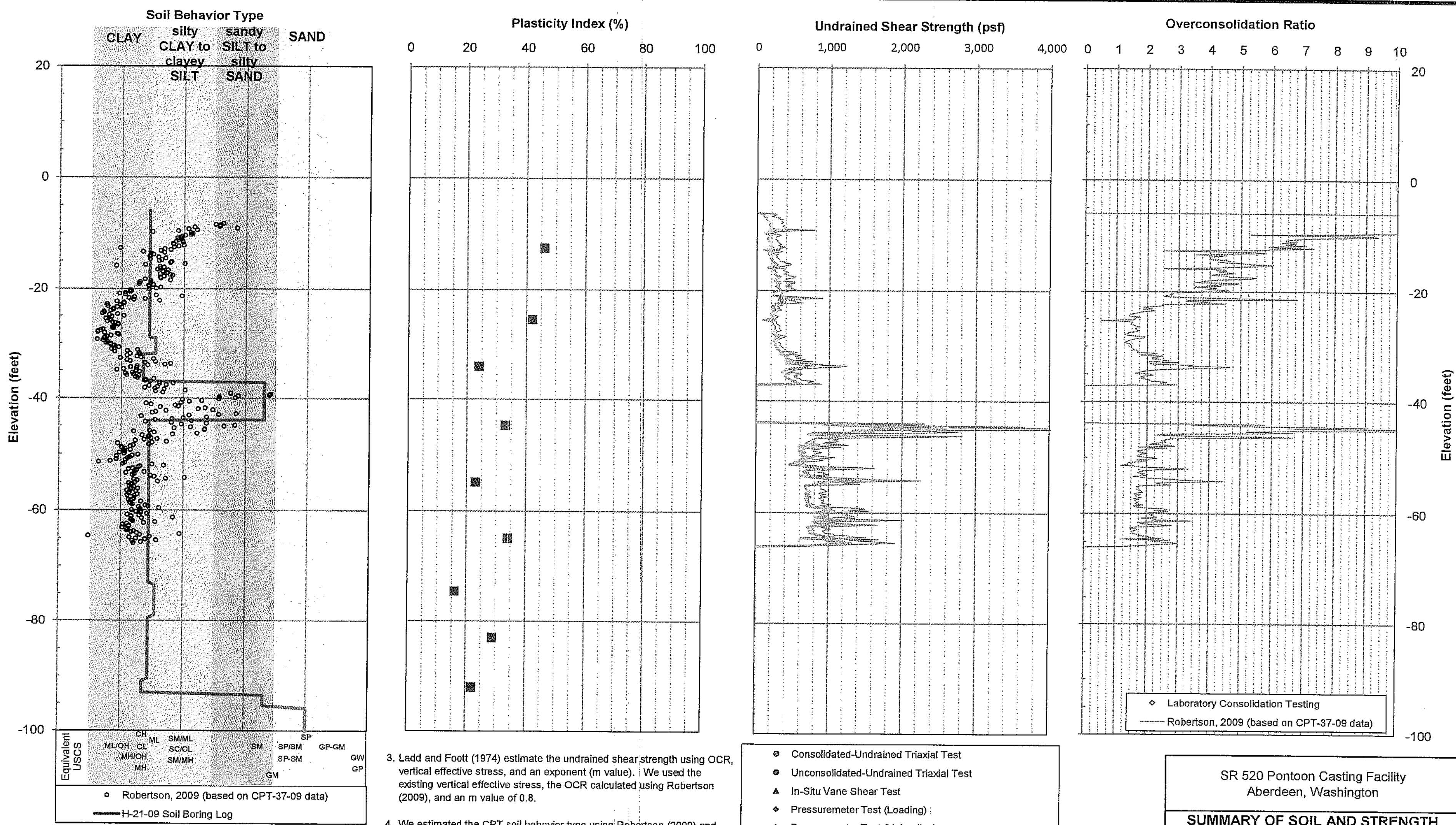
FIG. D-40



NOTES

1. Laboratory testing and in-situ testing was performed for samples obtained from boring H-21-09, which is in relatively close proximity to CPT-37-09.
2. We used Robertson (2009) to estimate the undrained shear strength and OCR using the existing vertical effective stress and data from CPT-37-09.
3. Ladd and Foott (1974) estimate the undrained shear strength using OCR, vertical effective stress, and an exponent (m value). We used the existing vertical effective stress, the OCR calculated using Robertson (2009), and an m value of 0.8.
4. We estimated the CPT soil behavior type using Robertson (2009) and data from CPT-37-09.
5. CPT = cone penetration test
OCR = overconsolidation ratio
psf = pounds per square foot

SR 520 Pontoon Casting Facility
Aberdeen, Washington

SUMMARY OF SOIL AND STRENGTH PARAMETERS
CPT-37-09 AND BORING H-21-09

February 2011 21-1-21190-015

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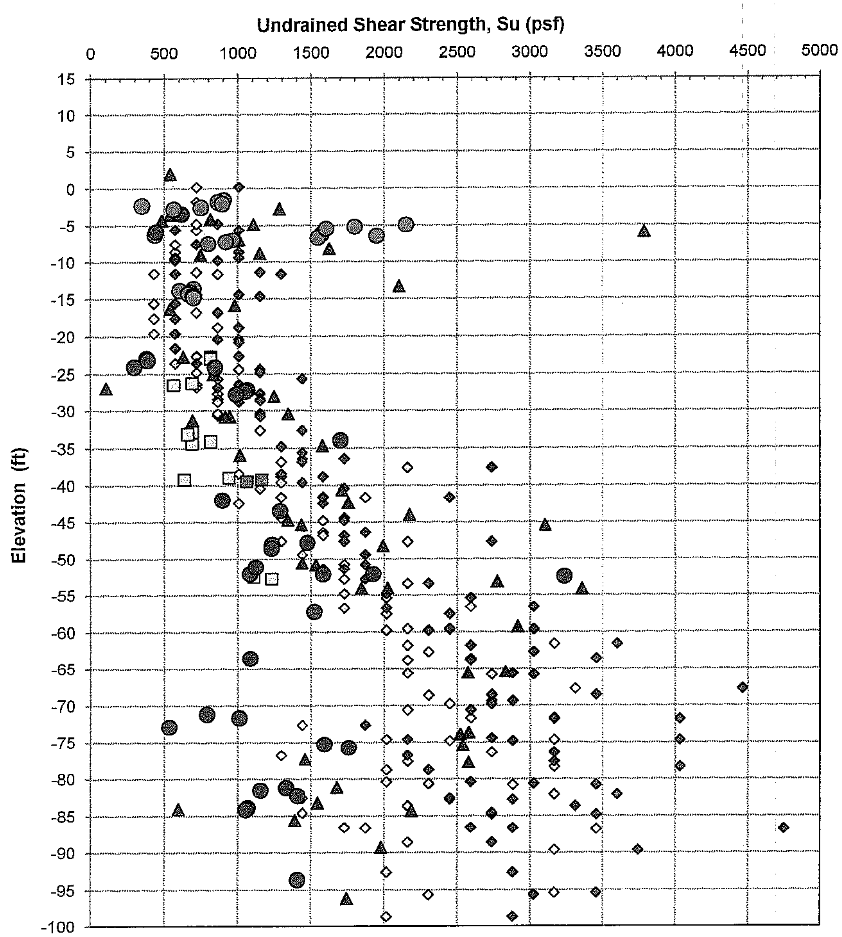
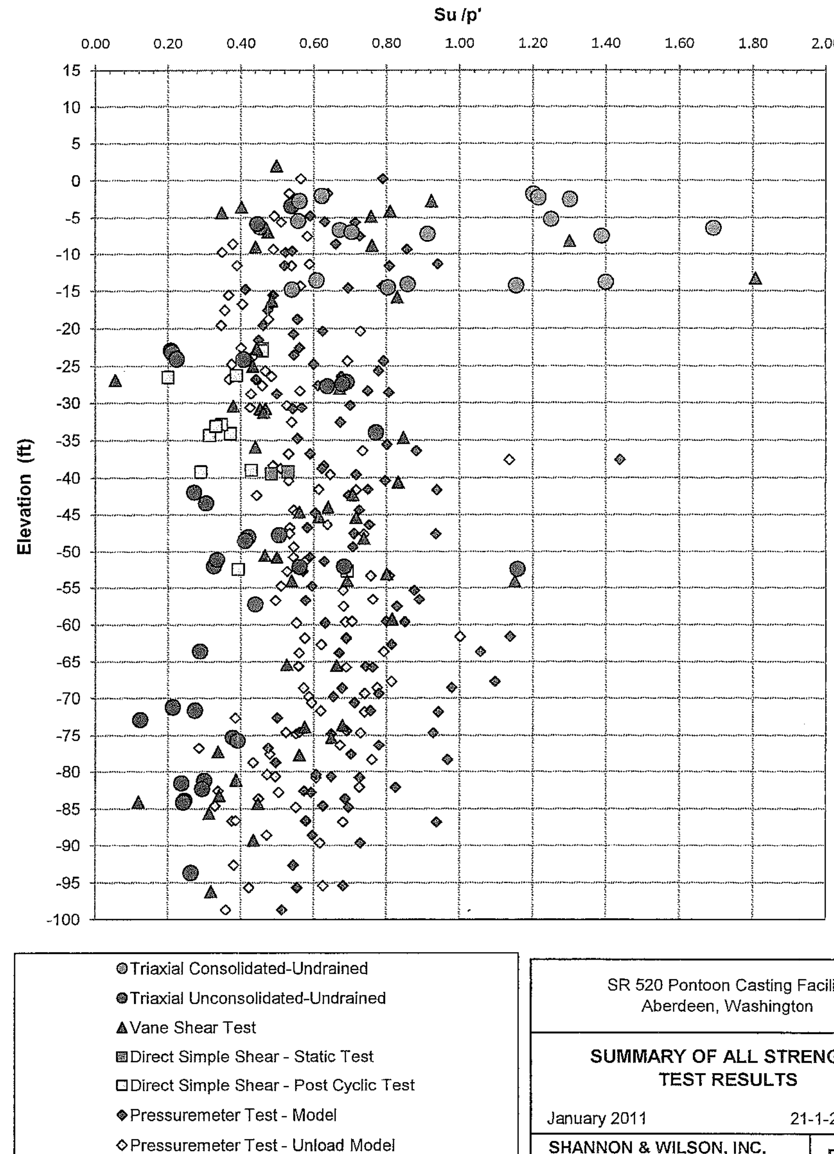
FIG. D-41

Test Summary.xlsm Printed: 1/5/2011 2:24 PM





SR 520 Pontoon Casting Facility
Aberdeen, Washington

SUMMARY OF ALL STRENGTH TEST RESULTS

January 2011 | 21-1-21190-015

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FIG. D-42

APPENDIX E

TEST PILE PROGRAM

APPENDIX E

TEST PILE PROGRAM

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FIGURES

ENCLOSURE

APPENDIX E

TEST PILE PROGRAM

E.1 INTRODUCTION

The test pile program consisted of driving five 24-inch-, two 18-inch-, and one 20-inch-diameter test piles with various wall thicknesses, ranging from ⅜ and ½ inch, and end conditions at two locations as shown in Figure 2 in the main text. A summary of the test pile program is shown in Table E-1.

Test piles were installed with a combination of a vibratory hammer and the Delmag D-46 diesel impact hammer. Piles were installed in segments that were welded together or connected with a mechanical collar. A mechanical collar splice is a compression splice that does not require welding. Photographs of the test piles installed with the two hammers are presented in Figure E-9. Photographs of the two connection type are presented in Figure E-10.

Each test pile was monitored with a pile driving analyzer (PDA) and analyzed with Case Pile Wave Analysis Program (CAPWAP) at the end of driving and for the three- and seven-day restrikes. The contractor used a Delmag D-62 pile-driving hammer to drive the test piles for the three- and seven-day restrikes. Robert Miner Dynamic Testing, under subcontract to Kiewit-General, performed the PDA monitoring and CAPWAP analyses for each phase of the test pile program. The CAPWAP analysis results are enclosed at the end of this appendix. A summary of the CAPWAP results is shown in Table E-2.

Pile driving resistance logs for each test pile are shown in Figures E-1 through E-8. Each pile driving resistance log plots the number of hammer blows to achieve 1 foot and 1 inch of penetration versus elevation. A plot of hammer stroke versus elevation is also shown. Photographs taken during test pile installation are shown in Figures E-9 through E-11.

Driving of each pile was terminated based on:

- The pile achieving the target driving resistance as determined by the initial resistance estimates.
- Discretion of the contractor due to:
  - Hammer limitations
  - Length of pile in the ground
  - Resistance achieved due to initial field estimates based on PDA monitoring

Test piles may be used as production piles if the estimated CAPWAP resistances satisfy design requirements.

E.2 SOUTH TEST PILE LOCATION

Three 24-inch-diameter test piles were driven at the south test pile location. The pile wall thickness ranged between 0.401 and ½ inch. The initial sections of the test piles were vibrated into the ground using a vibratory hammer. The approximate penetration of the test piles using the vibratory hammer ranged from approximate elevation -9 to -40 feet mean lower low water (MLLW) for the closed-end piles and -67 feet MLLW for the open-end pile. The test piles were driven to bearing elevation using a Delmag D-46 pile driving hammer. The final elevation of the two closed-end test piles was -131 feet MLLW and the open-end test pile was -144 feet MLLW.

Test pile P-2 could not be driven into the ground using the vibratory hammer, likely due to hard driving conditions or obstructions in the upper approximately 10 to 15 feet of fill. The Contractor excavated an approximate 10-foot-deep trench and removed dense sand and gravel, logs, and soft silt and sand from the excavation. The Contractor then placed the first segment of test pile P-2 at the base of the trench and vibrated the pile to approximate elevation -9 feet MLLW. The sand, gravel, and silt were placed back in the trench following vibratory pile driving.

The pile segments of test piles P-1 and P-3 were welded, while the pile segments of test pile P-2 were spliced with a mechanical collar. While driving test pile P-2, the third pile segment and the mechanical collar separated from the second pile segment when the mechanical splice was slightly above the ground surface. The top of the second segment of P-2 was damaged and subsequently removed. The collar and the third segment were replaced onto the top of the second segment of P-2 and driving was resumed.

The use of a mechanical splice is not permitted for final design according to Washington State Department of Transportation (WSDOT) Standard Specifications. Test pile P-2 was not used to determine the pile driving parameters for final design.

Test piles P-1 and P-2 were driven to pile driving blow counts greater than 100 blows per foot near final initial driving of the test piles. Pile driving blow counts greater than 100 blows per foot is not permitted for final design according to WSDOT Standard Specifications. The CAPWAP results show maximum compressive stresses in these test piles of about 30 kips per square inch (ksi), which is less than 90 percent of the yield strength of the steel. Yielding of the pile tops was not observed during driving so the PDA/CAPWAP results for test pile P-1 were considered when determining the pile driving parameters for final design. At the seven-day restrike, the CAPWAP results for test pile P-2 show a maximum compressive stress at the top of the pile greater than 90 percent of the yield stress of the steel. Yielding of the pile top was not observed during the seven-day restrike.

Test piles P-1 through P-3 were driven as test piles and will not be used for permanent structure support. The production piles will be driven in accordance with the WSDOT Standard Specifications.

E.3 NORTH TEST PILE LOCATION

At the north test pile location, the Contractor attempted to drive the initial sections of the test piles with the vibratory hammer. After several (greater than about 12) unsuccessful attempts to drive an open-end pipe pile more than a few feet into the ground, the Contractor used the Delmag D-46 pile-driving hammer to drive the initial segment of the test piles. The test piles were driven to bearing elevation using a Delmag D-46 pile-driving hammer. The final elevation of the closed-end piles ranged from -102 to -125 feet MLLW and -121 to -125 feet MLLW for the open-end piles.

The pile segments of test piles P-5 through P-8 were welded, while the pile segments of test pile P-4 were spliced with a mechanical collar. Test pile P-4 was not used to determine the pile driving parameters for final design. During driving, test pile P-8 encountered an obstruction at approximately 15 feet below ground surface which slightly redirected the pile tip and caused the pile to be installed at a slight batter.

Yielding of the top of the pile occurred during final driving, three-day, and/or seven-day restrike at several North test piles. In this case, the yielded section was generally removed and a new 18-inch section was welded to the top of the pile. Figure E-11 shows a yielded pile and a pile modified with the added section. Pile yielding and subsequent modification occurred at P-4 during three-day restrike, at P-7 during seven-day restrike, and P-8 during final drive, three-day, and seven-day restrikes. Tops of test piles P-5 and P-6 also yielded during seven-day restrike.

The CAPWAP results show a maximum compressive stress at the top of test pile P-4 of 38.1 ksi, which is less than 90 percent of the yield strength of the steel. The steel at the top of P-4 may have yielded for various reasons, including potential misalignment during driving of the pile and hammer which could have caused a stress concentration on one side of the pile.

At the seven-day restrike, the CAPWAP results for test pile P-6 show a maximum compressive stress at the top of the pile greater than 90 percent of the yield stress of the steel.

Test piles P-4 through P-8 were driven as test piles and will not be used for permanent structure support. The production piles will be driven in accordance with the WSDOT Standard Specifications.

E.4 CASE PILE WAVE ANALYSIS PROGRAM (CAPWAP) RESULTS

For each test pile, the CAPWAP results were separated into three sections based on the soil type encountered in nearby borings, trends in PDA measurements, and engineering judgment. An average, minimum, and maximum unit side resistance and incremental shaft resistance was estimated from the CAPWAP results for each soil unit for the three- and seven-day re-strikes. The unit end resistance was estimated at the end of initial driving for test piles P-1 through P-7 and at the end of initial driving and three-day re-drive for test pile P-8. The shaft and toe CAPWAP resistance for the end of initial driving and the three- and seven-day re-strikes were also estimated. A summary of the PDA/CAPWAP results is shown in Table E-2.

TABLE E-1
SUMMARY OF TEST PILE PROGRAM

Location	Test Pile	Pile Diameter (inches)	Pile Wall Thickness (inches)	Approximate Pile Length (feet)	Pile End Condition	Splice Method	Nearby Boring	Date	Drive Condition	Hammer Type	Pile Driving Blow Count (blow/foot)	Approximate Tip Elevation (feet)
South	P-1	24	0.5	142	Closed	Weld	H-14P-09 and H-4-08	4/13/2010	EOV	-	N/A	-40.0
								4/15/2010	EOID	D-46	53	-128.0
											88	-129.0
											122	-130.0
											174	-131.0
											67 / 4 inch	-131.3
								4/19/2010	Three-day Restrike	D-62	20 / 1 inch	-131.4
								4/26/2010	Seven-day Restrike	D-62	35 / 1 inch	-131.5
											29 / 1 inch	-131.6
	P-2	24	0.401	142	Closed	Mechanical Collar	H-14P-09 and H-4-08	4/15/2010	EOV	-	N/A	-9.0
								4/15/2010	EOID	D-46	36	-128.0
											54	-129.0
											122	-130.0
											162	-131.0
								4/19/2010	Three-day Restrike	D-62	23 / 1 inch	-131.1
								4/26/2010	Seven-day Restrike	D-62	~25 / 1 inch [1]	-131.2
											23 / 1 inch	-131.3
	P-3	24	0.401	155	Open	Weld	H-14P-09 and H-4-08	4/13/2010	EOV	-	N/A	-67.0
								4/15/2010	EOID	D-46	46	-139.0
											56	-140.0
											60	-141.0
											71	-142.0
											65 / 10 inches	-142.8
								4/19/2010	Three-day Restrike	D-62	15 / 1 inch	-142.9
											6 / 1 inch	-143.0
								4/26/2010	Seven-day Restrike	D-62	24 / 1 inch	-143.1
											17 / 1 inch	-143.2
											11 / 1 inch	-143.3
											11 / 1 inch	-143.3
											97 / 8 inches	-144.0
											44 / 6 inches	-144.5

TABLE E-1
SUMMARY OF TEST PILE PROGRAM

Location	Test Pile	Pile Diameter (inches)	Pile Wall Thickness (inches)	Approximate Pile Length (feet)	Pile End Condition	Splice Method	Nearby Boring	Date	Drive Condition	Hammer Type	Pile Driving Blow Count (blow/foot)	Approximate Tip Elevation (feet)
North	P-4	24	0.401	140	Closed	Mechanical Collar	BH-1-10 and H-11P-09	Vibratory hammer not used				
								4/22/2010	EOID	D-46	57	-121.0
											61	-122.0
											66	-123.0
											75	-124.0
											74	-125.0
								4/26/2010	Three-day Restrike	D-62	14 / 1 inch	-125.1
											9 / 0.5 inch	-125.1
								5/3/2010	Seven-day Restrike	D-62	26 / 1 inch	-125.2
											18 / 1 inch	-125.3
											12 / 1 inch	-125.4
	P-5	24	0.401	140	Open	Weld	BH-1-10 and H-11P-09	Vibratory hammer not used				
								4/21/2010	EOID	D-46	19	-121.0
											22	-122.0
											22	-123.0
											25	-124.0
											26	-125.0
								4/26/2010	Three-day Restrike	D-62	15 / 1 inch	-125.1
											11 / 1 inch	-125.2
											6 / 1 inch	-125.3
								5/3/2010	Seven-day Restrike	D-62 [2]	32 / 0.5 inch	-125.3
	P-6	18	0.375	118	Closed	Weld	BH-1-10 and H-11P-09	Vibratory hammer not used				
								4/21/2010	EOID	D-46	17	-99.0
											50	-100.0
											56	-101.0
											54	-102.0
											25 / 6 inches	-102.5
								4/26/2010	Three-day Restrike	D-62	12 / 1 inch	-102.6
											8 / 1 inch	-102.7
								5/3/2010	Seven-day Restrike	D-62 [2]	17 / 1 inch	-102.8
											17 / 1 inch	-102.8
											15 / 1 inch	-102.9
											11 / 1 inch	-103.0

TABLE E-1
SUMMARY OF TEST PILE PROGRAM

Location	Test Pile	Pile Diameter (inches)	Pile Wall Thickness (inches)	Approximate Pile Length (feet)	Pile End Condition	Splice Method	Nearby Boring	Date	Drive Condition	Hammer Type	Pile Driving Blow Count (blow/foot)	Approximate Tip Elevation (feet)
North	P-7	18	0.375	136	Open	Weld	BH-1-10 and H-11P-09	Vibratory hammer not used				
								4/21/2010	EOID	D-46	19	-117.0
											19	-118.0
											19	-119.0
											16	-120.0
											14 / 10 inches	-120.8
								4/26/2010	Three-day Restrike	D-62	6 / 1 inch	-120.9
											3 / 1 inch	-121.0
											2 / 1 inch	-121.1
								5/3/2010	Seven-day Restrike	D-62 [3]	23 / 1 inch	-121.2
											20 / 1 inch	-121.3
											13 / 1 inch	-121.3
	P-8	20	0.375	126	Closed	Weld	BH-1-10 and H-11P-09	Vibratory hammer not used				
								4/22/2010	EOID	D-46	72	-101.0
											78	-102.0
											69	-103.0
											67	-104.0
											54	-105.0
								4/26/2010	Three-day Restrike	D-62	8 / 1 inch	-105.1
											6 / 1 inch	-105.2
											44 / 10 inches	-106.0
											58	-107.0
											50	-108.0
											49	-109.0
											47	-110.0
											20 / 6 inches [2]	-110.5
								5/3/2010	Seven-day Restrike	D-62 [2]	13 / 1 inch	-110.6
											8 / 1 inch	-110.7
											8 / 1 inch	-110.75

Notes:
1. Insufficient fuel delivery to hammer resulted in a few weak hammer blows.
2. Top of pile yielded.
3. Fuel setting was reduced from 4 to 3 after the first three blows in an effort to avoid yielding the top of the pile.

EOID = End of Initial Driving; EOV = End of Vibration; N/A = Not Applicable

TABLE E-2
SUMMARY OF PDA/CAPWAP RESULTS

Location	Test Pile	Pile Diameter (inches)	Pile Wall Thickness (inches)	Approximate Pile Length (feet)	Approximate Tip Elevation (feet)	Pile End Condition	Splice Method	Nearby Boring	Drive Condition	Pile Driving Blow Count at Average Stroke Height	Approximate Elevation (feet)	Soil Type	Unit Side Resistance, fs (ksf)			Unit End Bearing, q_{ult} (ksf)	Incremental Shaft Resistance (kips)	CAPWAP Resistance (kips)	
													Min	Max	Average			Shaft	Toe
South	P-1	24	0.5	142	-131.6	Closed	Weld	H-14P-09 and H-4-08	EOID	67 / 4 inch @ 9.2'	-131	Very dense sand/gravel	-			220	-	70	690
									Three-day Restrike	20 / 1 inch	10 - -100	Soft Soil	0.1	0.7	0.3	191	134	330	600
											-100 - -125	Medium to very dense sand/gravel	0.6	0.7	0.6		141		
											-125 - -131	Very dense sand/gravel	0.9	1.3	1.1		55		
									Seven-day Restrike	35 / 1 inch	10 - -100	Soft Soil	0.0	0.9	0.4	76	238	660	240
											-100 - -125	Medium to very dense sand/gravel	0.9	1.7	1.3		293		
											-125 - -131.3	Very dense sand/gravel	2.4	3.1	2.7		130		
	P-2	24	0.401	142	-131.3	Closed	Mechanical Collar	H-14P-09 and H-4-08	EOID	162 bpf @ 9.2'	-131	Very dense sand/gravel	-			178	-	40	560
									Three-day Restrike	23 / 1 inch	10 - -100	Soft Soil	0.0	0.4	0.1	111	73	340	350
											-100 - -125	Medium to very dense sand/gravel	0.4	0.9	0.6		171		
											-125 - -131	Very dense sand/gravel	1.5	2.3	1.9		96		
									Seven-day Restrike	~25 / 1 inch[1]	10 - -100	Soft Soil	0.0	0.8	0.2	80	131	480	250
											-100 - -125	Medium to very dense sand/gravel	1.0	1.2	1.1		256		
											-125 - -131	Very dense sand/gravel	1.6	2.2	1.9		93		
	P-3	24	0.401	155	-144.5	Open	Weld	H-14P-09 and H-4-08	EOID	65 / 10 inch @ 9.2'	-143	Very dense sand/gravel	-			19	-	440	60
									Three-day Restrike	15 / 1 inch	10 - -100	Soft Soil	0.0	0.8	0.3	29	198	530	90
											-100 - -125	Medium to very dense sand/gravel	0.4	0.6	0.5		80		
											-125 - -142.8	Very dense sand/gravel	1.0	2.8	2.0		253		
									Seven-day Restrike	24 / 1 inch	10 - -100	Soft Soil	0.0	1.1	0.4	29	270	561	90
											-100 - -125	Medium to very dense sand/gravel	0.2	0.5	0.3		51		
											-125 - -143	Very dense sand/gravel	1.4	2.4	1.9		240		

TABLE E-2
SUMMARY OF PDA/CAPWAP RESULTS

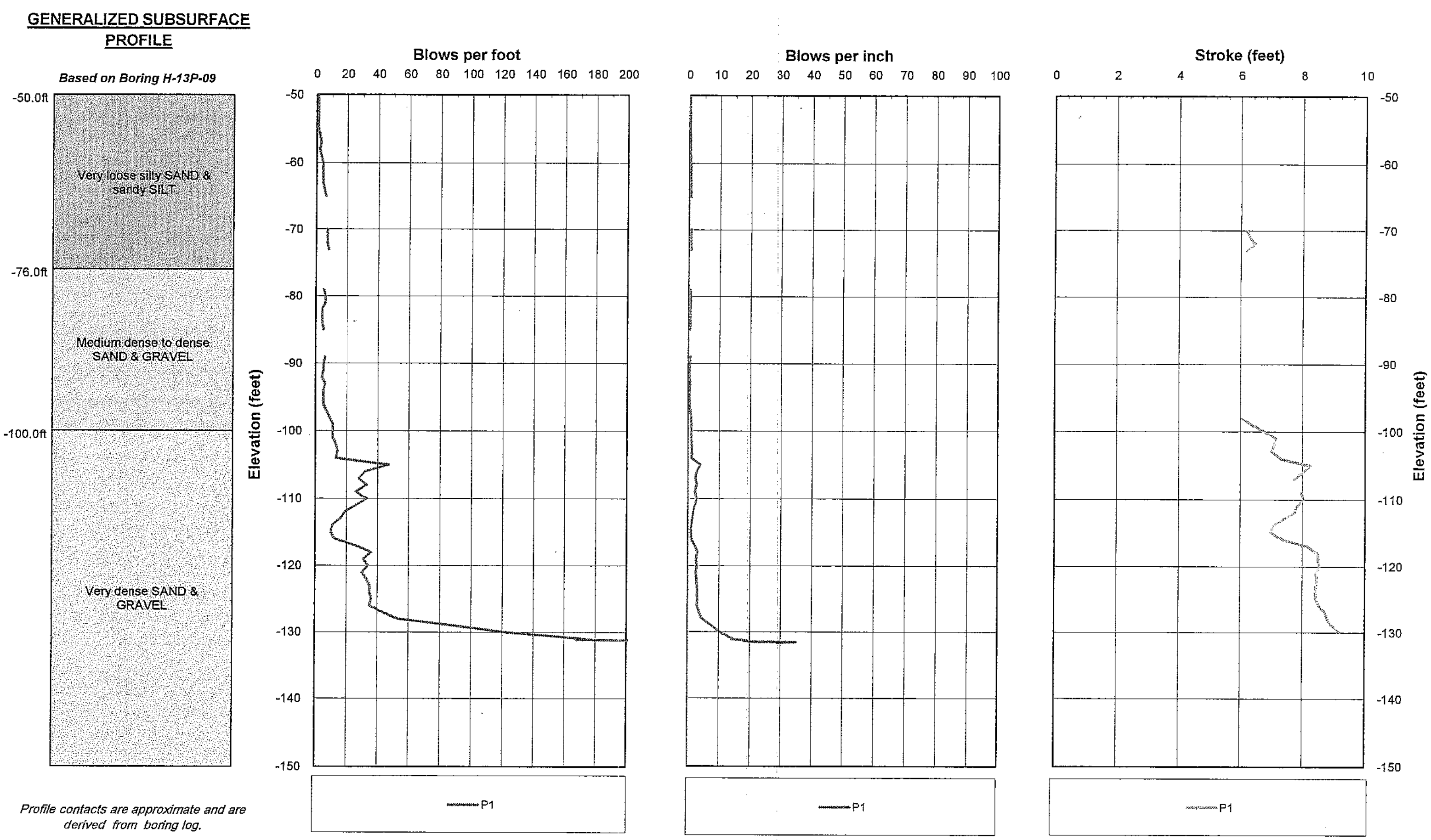
Location	Test Pile	Pile Diameter (inches)	Pile Wall Thickness (inches)	Approximate Pile Length (feet)	Approximate Tip Elevation (feet)	Pile End Condition	Splice Method	Nearby Boring	Drive Condition	Pile Driving Blow Count at Average Stroke Height	Approximate Elevation (feet)	Soil Type	Unit Side Resistance, fs (ksf)			Unit End Bearing, q_{ult} (ksf)	Incremental Shaft Resistance (kips)	CAPWAP Resistance (kips)	
													Min	Max	Average			Shaft	Toe
North	P-4	24	0.401	140	-125	Closed	Mechanical Collar	BH-1-10 and H-11P-09	EOID	74 bpf @ 8.8'	-125	Very dense sand/gravel	-			166	-	90	520
									Three-day Restrike	14 / 1 inch	15 - -95	Soft Soil	0.2	0.7	0.4	86	277	780	270
											-95 - -115	Medium to very dense sand/gravel	0.7	1.9	1.2		235		
											-115 - -125	Very dense sand/gravel	2.7	3.3	3.0		268		
									Seven-day Restrike	26 / 1 inch	15 - -95	Soft Soil	0.2	0.9	0.5	48	337	850	150
											-95 - -115	Medium to very dense sand/gravel	1.0	1.5	1.2		204		
											-115 - -125	Very dense sand/gravel	2.3	4.2	3.3		309		
	P-5	24	0.401	140	-125	Open	Weld	BH-1-10 and H-11P-09	EOID	26 bpf @ 8.2'	-125	Very dense sand/gravel	-			105	-	166	330
									Three-day Restrike	15 / 1 inch	15 - -95	Soft Soil	0.1	1.0	0.5	83	349	590	260
											-95 - -115	Medium to very dense sand/gravel	0.8	1.0	0.9		110		
											-115 - -125	Very dense sand/gravel	1.5	1.7	1.6		131		
									Seven-day Restrike	32 / 0.5 inch [2]	15 - -95	Soft Soil	0.3	1.2	0.7	73	481	770	230
											-95 - -115	Medium to very dense sand/gravel	1.2	1.4	1.3		162		
											-115 - -125	Very dense sand/gravel	1.5	1.5	1.5		128		
	P-6	18	0.375	118	-103	Closed	Weld	BH-1-10 and H-11P-09	EOID	15 / 6 inch @ 8.3'	-103	Medium to very dense sand/gravel	-			243	-	70	430
									Three-day Restrike	12 / 1 inch	15 - -85	Soft Soil	0.0	1.0	0.3	130	193	440	230
											-85 - -103	Medium to very dense sand/gravel	1.4	3.4	2.3		247		
												Very dense sand/gravel	-	-	-		-		
									Seve-day Restrike	17 / 1 inch	15 - -85	Soft Soil	0.2	1.8	0.6	221	309	530	390
											-85 - -103	Medium to very dense sand/gravel	2.1	2.6	2.3		221		
											-	Very dense sand/gravel	-	-	-		-		

TABLE E-2
SUMMARY OF PDA/CAPWAP RESULTS

Location	Test Pile	Pile Diameter (inches)	Pile Wall Thickness (inches)	Approximate Pile Length (feet)	Approximate Tip Elevation (feet)	Pile End Condition	Splice Method	Nearby Boring	Drive Condition	Pile Driving Blow Count at Average Stroke Height	Approximate Elevation (feet)	Soil Type	Unit Side Resistance, fs (ksf)			Unit End Bearing, q_{ult} (ksf)	Incremental Shaft Resistance (kips)	CAPWAP Resistance (kips)	
													Min	Max	Average			Shaft	Toe
North	P-7	18	0.375	136	-121	Open	Weld	BH-1-10 and H-11P-09	EOID	14 / 10 inch @ 7.8'	-121	Very dense sand/gravel	-			124	-	100	220
									Three-day Restrike	6 / 1 inch	15 - -85	Soft Soil	0.2	1.7	0.8	164	323	504	290
											-85 - -121	Medium to very dense sand/gravel	0.4	1.2	0.8		181		
											-	Very dense sand/gravel	-	-	-		-		
									Seven-day Restrike	23 / 1 inch[3]	15 - -85	Soft Soil	0.2	1.4	0.7	147	326	590	260
											-85 - -121	Medium to very dense sand/gravel	1.0	2.0	1.5		264		
											-	Very dense sand/gravel	-	-	-		-		
	P-8	20	0.375	126	-111	Closed	Weld	BH-1-10 and H-11P-09	EOID	54 bpf @ 8.6'	-105	Medium to very dense sand/gravel	-			170	-	150	370
									Three-day Restrike	8 / 1 inch	15 - -85	Soft Soil	0.1	1.0	0.4	170	199	340	370
											-85 - 105	Medium to very dense sand/gravel	0.9	1.1	1.0		141		
											-	Very dense sand/gravel	-	-	-		-		
									EOR	20 / 6 inches	-111	Medium to very dense sand/gravel	-			206	-	150	450
									Seven-day Restrike	13 / 1 inch	15 - -85	Soft Soil	0.1	1.0	0.5	128	240	600	280
											-85 - -111	Medium to very dense sand/gravel	1.3	3.4	2.4		360		
											-	Very dense sand/gravel	-	-	-		-		

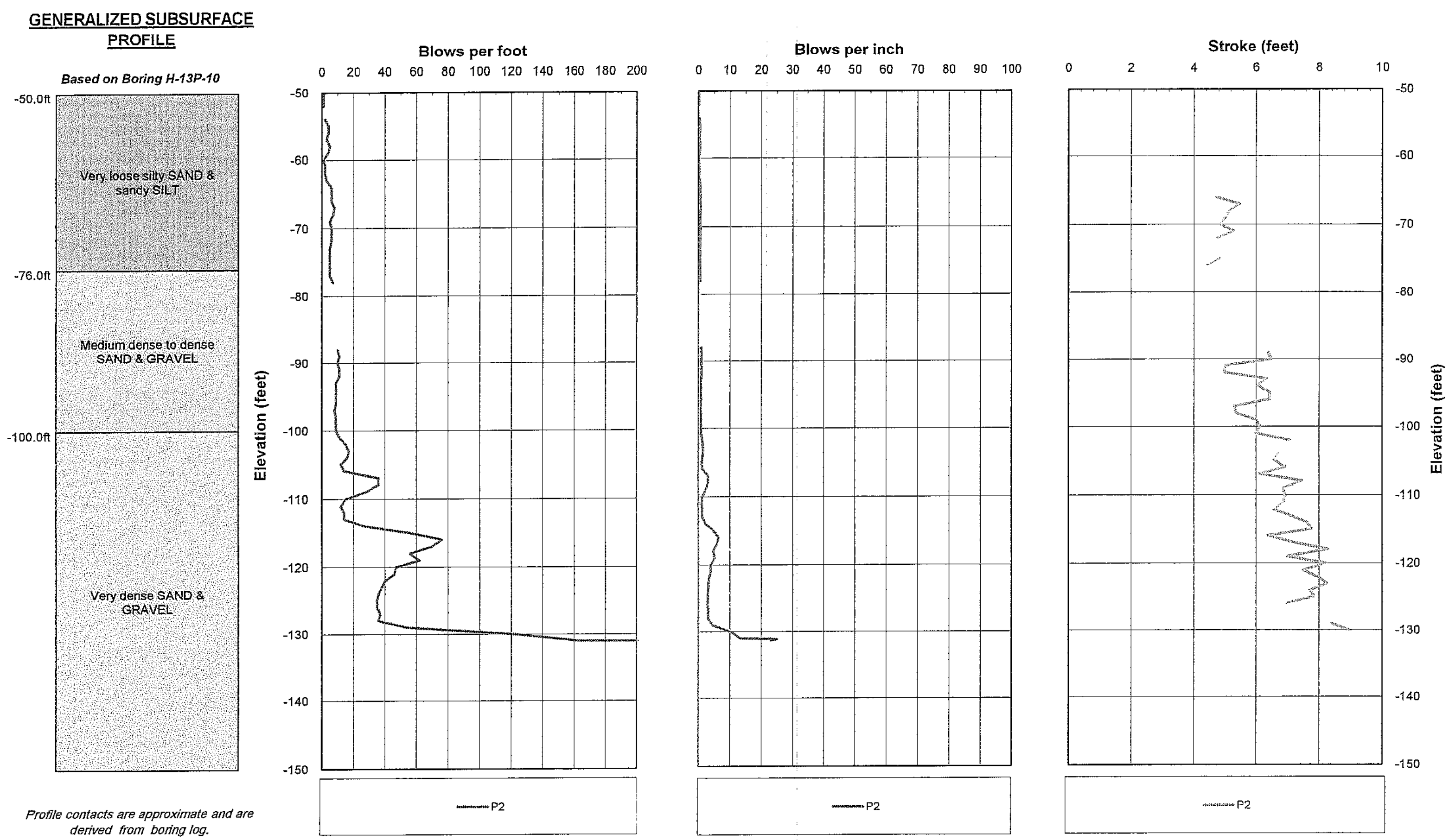
Notes:
1. Insufficient fuel delivery to hammer resulted in a few weak hammer blows.
2. Top of pile yielded.
3. Fuel setting was reduced from 4 to 3 after the first three blows in an effort to avoid yielding the top of the pile.

bpf = blows per foot
CAPWAP = Case Pile Wave Analysis Program
EOID = End of Initial Driving
EOV = End of Vibration
N/A = Not Applicable
PDA = pile driving analyzer



Notes:
1. Pile driven to end of initial driving using a Delmag D-46 hammer.
2. Pile re-strike was accomplished using a Delmag D-62 hammer.

SR 520 Pontoon Casting Facility Aberdeen, Washington
PILE DRIVING RESISTANCE **P1 (SOUTH)** **24 X 1/2-INCH, CLOSED-END**
May 2010 21-1-21190-015
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants — **FIG. E-1**



Notes:
1. Pile driven to end of initial driving using a Delmag D-46 hammer.
2. Pile re-strike was accomplished using a Delmag D-62 hammer.

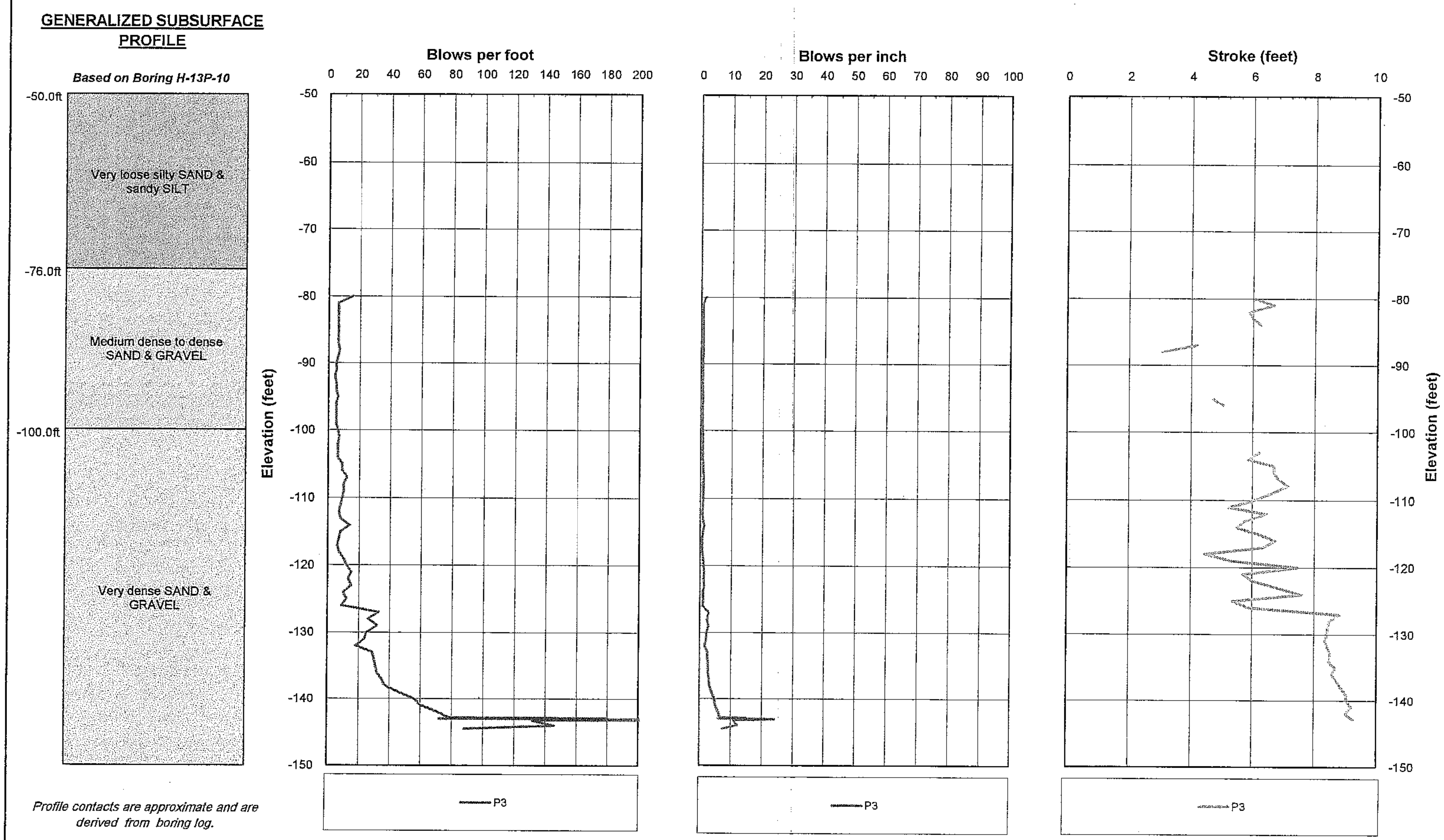
SR 520 Pontoon Casting Facility
Aberdeen, Washington

PILE DRIVING RESISTANCE
P2 (SOUTH)
24 X 0.401-INCH, CLOSED-END

May 2010 | 21-1-21190-015

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. E-2



Profile contacts are approximate and are derived from boring log.

Notes:
1. Pile driven to end of initial driving using a Delmag D-46 hammer.
2. Pile re-strike was accomplished using a Delmag D-62 hammer.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PILE DRIVING RESISTANCE
P3 (SOUTH)
24 X 0.401-INCH, OPEN-END

May 2010 | 21-1-21190-015

SHANNON & WILSON, INC.
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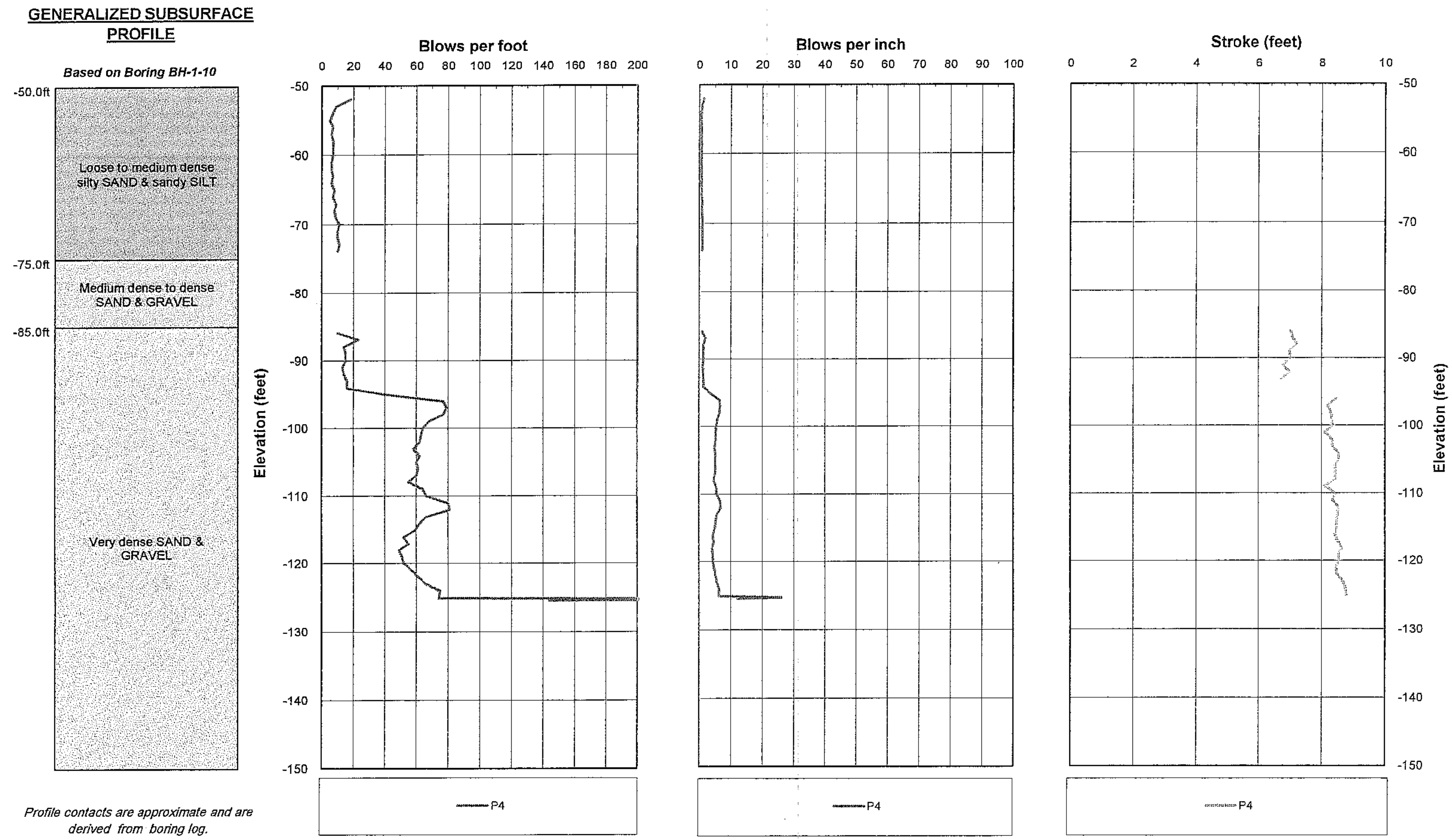
FIG. E-3



Notes:
1. Pile driven to end of initial driving using a Delmag D-46 hammer.
2. Pile re-strike was accomplished using a Delmag D-62 hammer.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PILE DRIVING RESISTANCE
P4 (NORTH)
24 X 0.401-INCH, CLOSED-END

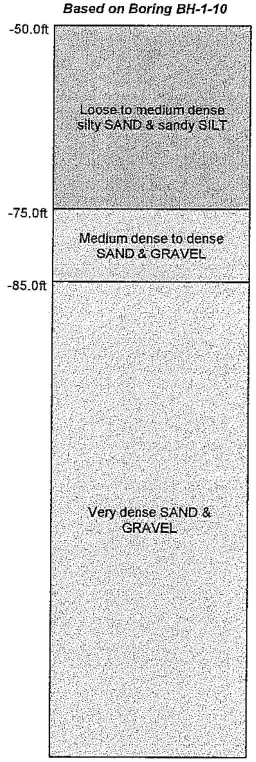
May 2010 | 21-1-21190-015

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

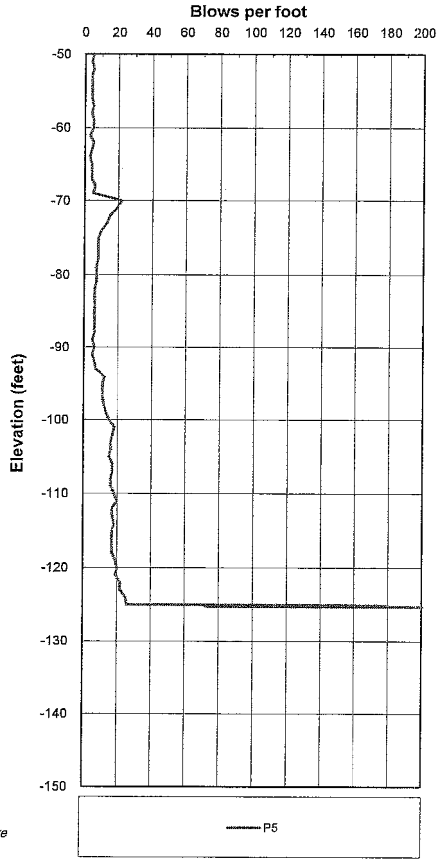
FIG. E-4

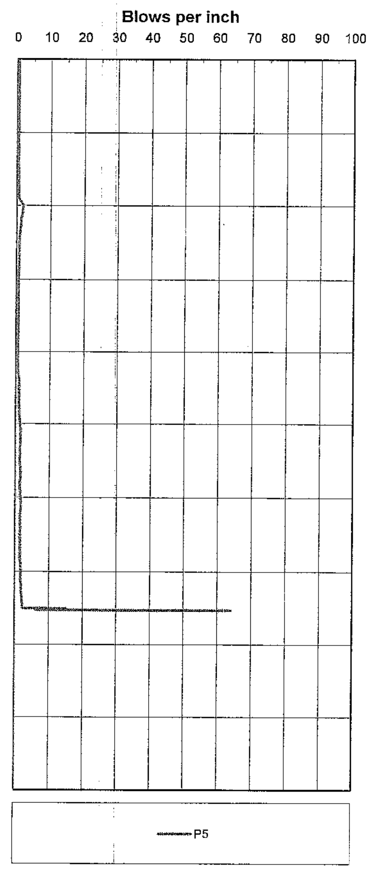
GENERALIZED SUBSURFACE PROFILE

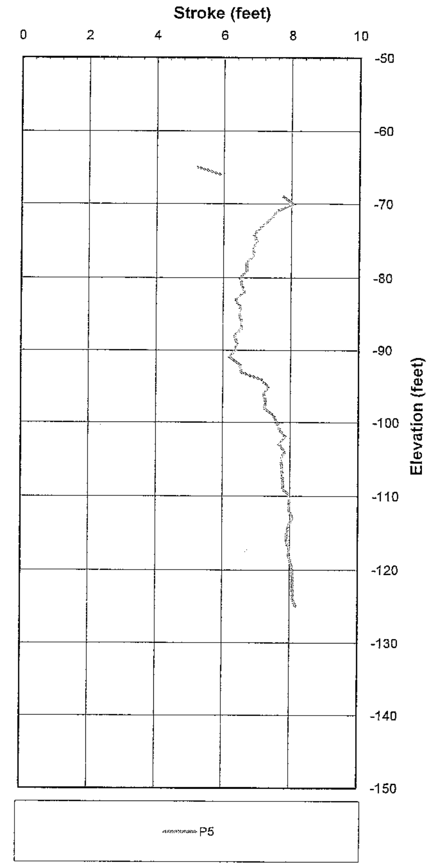
Based on Boring BH-1-10



Profile contacts are approximate and are derived from boring log.







Notes:
1. Pile driven to end of initial driving using a Delmag D-46 hammer.
2. Pile re-strike was accomplished using a Delmag D-62 hammer.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PILE DRIVING RESISTANCE
P5 (NORTH)
24 X 0.401-INCH, OPEN-END

May 2010 | 21-1-21190-015

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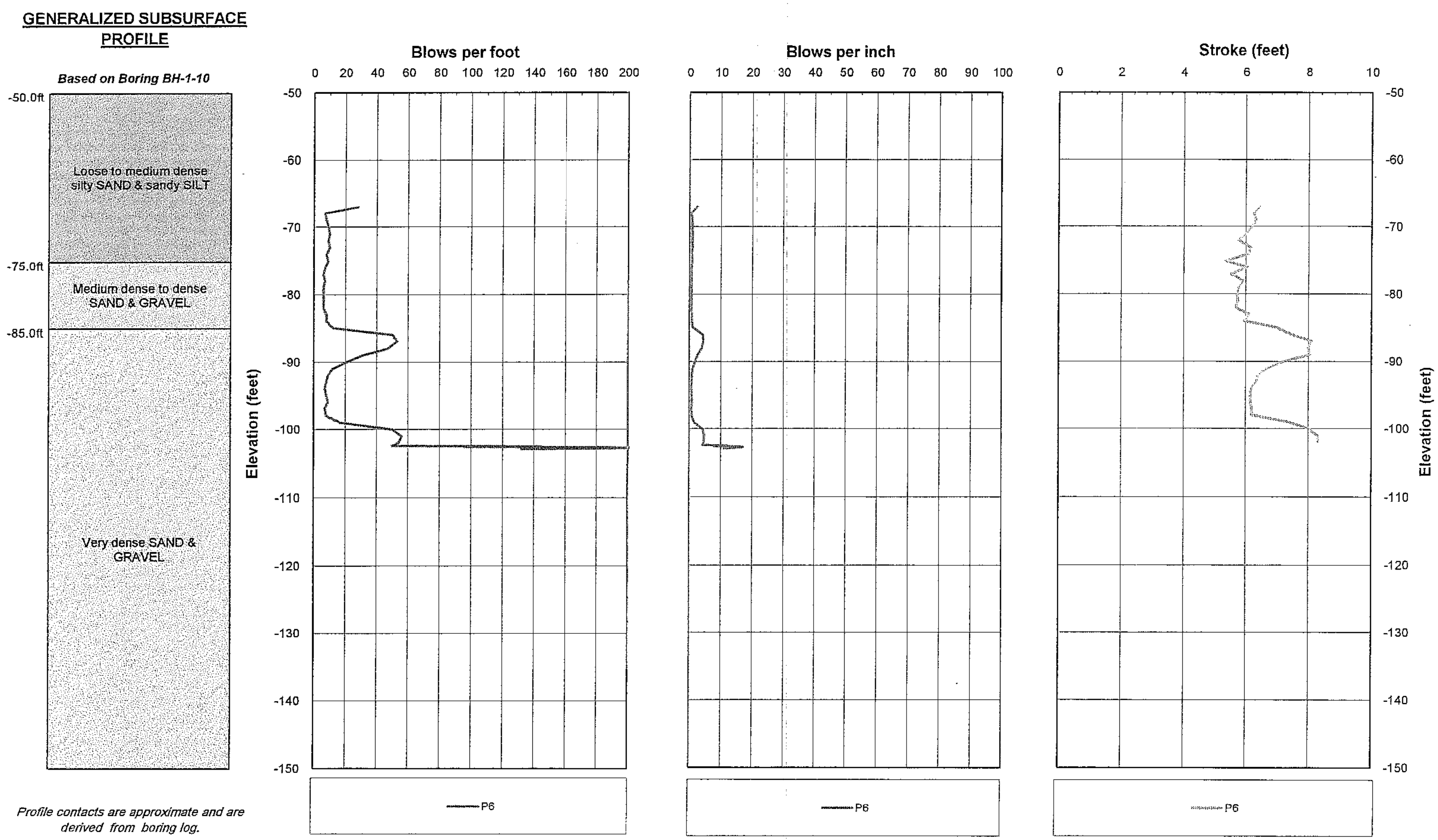
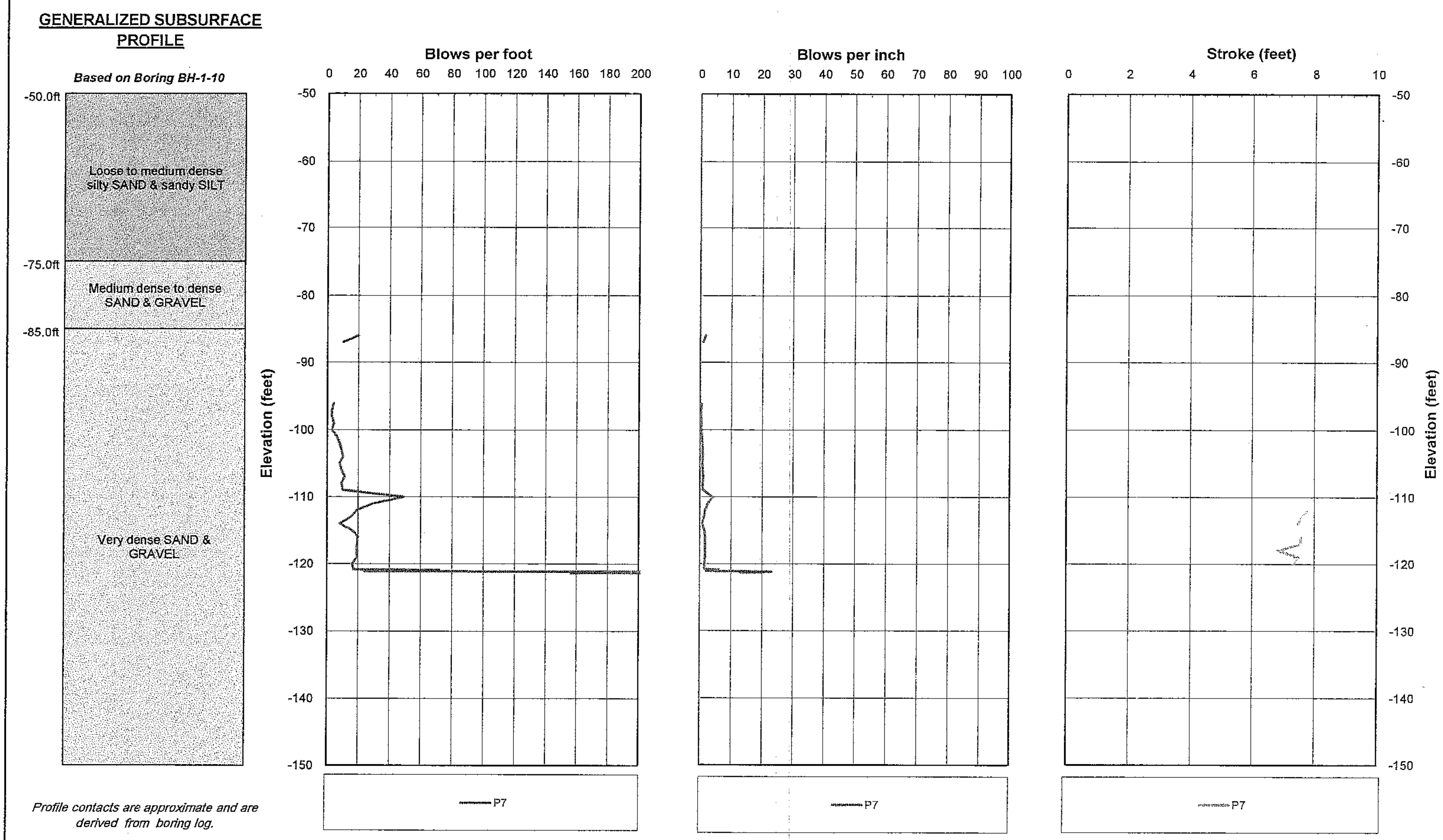
FIG. E-5



Notes:

1. Pile driven to end of initial driving using a Delmag D-46 hammer.
2. Pile re-strike was accomplished using a Delmag D-62 hammer.

SR 520 Pontoon Casting Facility Aberdeen, Washington
PILE DRIVING RESISTANCE **P6 (NORTH)** **18 X 3/8-INCH, CLOSED-END**
May 2010 — 21-1-21190-015
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants — **FIG. E-6**



Notes:
1. Pile driven to end of initial driving using a Delmag D-46 hammer.
2. Pile re-strike was accomplished using a Delmag D-62 hammer.
3. The Delmag D-62 hammer was set to fuel energy 4 for the first re-strick and to fuel energy 3 for the second re-strike.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PILE DRIVING RESISTANCE
P7 (NORTH)
18 X 3/8-INCH, OPEN-END

May 2010 | 21-1-21190-015

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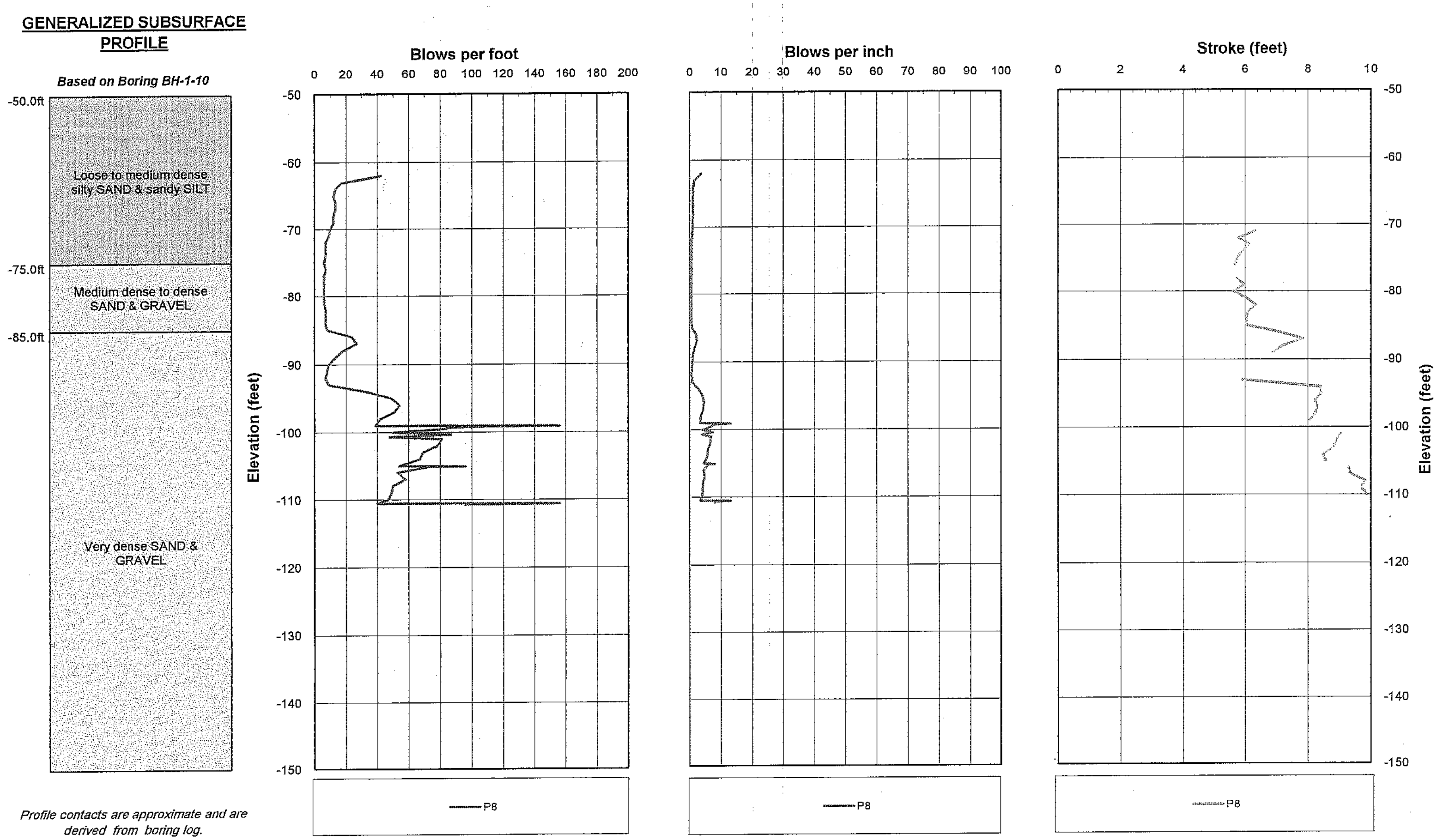
FIG. E-7



Notes:
1. Pile driven to end of initial driving using a Delmag D-46 hammer.
2. Pile re-strike was accomplished using a Delmag D-62 hammer.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PILE DRIVING RESISTANCE
P8 (NORTH)
20 X 3/8-INCH, CLOSED-END

May 2010 | 21-1-21190-015

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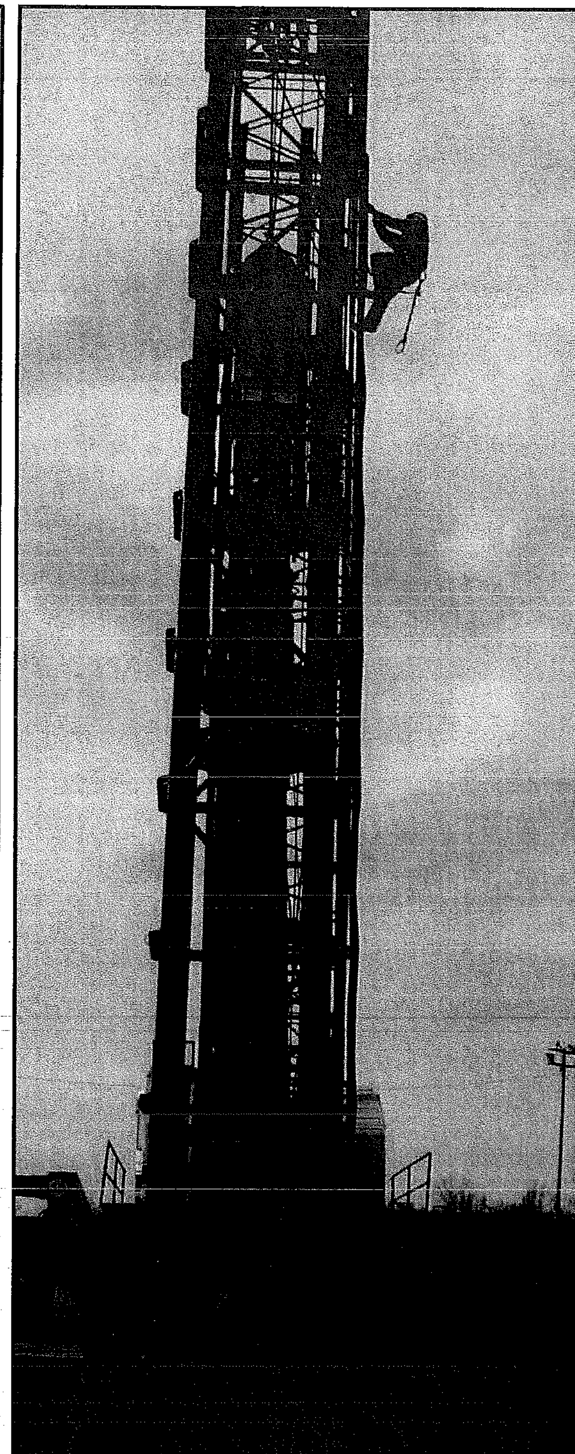
FIG. E-8



Notes

Left: Test Pile installed using a vibratory hammer.

Right: Test Pile installed using a D-46 pile driving hammer.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

TEST PILE INSTALLATION

August 2010 | 21-1-21190-015

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FIG. E-9

Notes

Top: Mechanical collar splice.

Bottom: Welded splice.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

TEST PILE SPLICE METHODS

August 2010 21-1-21190-015

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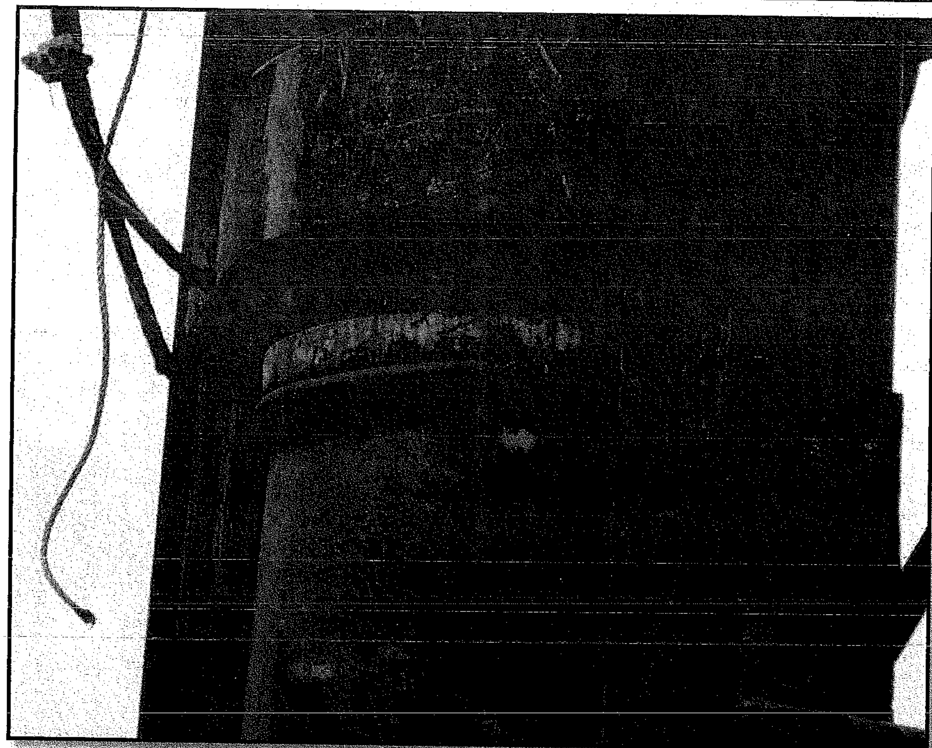
FIG. E-10



Notes

Top: Top of pile yielded during re-srike.

Bottom: Yielded portion of pile was removed and an additional pile segment was welded over about the top 18 inches of the pile.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

TEST PILE RE-STRIKE

August 2010 | 21-1-21190-015

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FIG. E-11

ENCLOSURE

REPORT BY ROBERT MINER DYNAMIC TESTING, INC.
DATED AUGUST 30, 2010

Summary of CAPWAP Results, 520 Pontoon Casting Yard, Test Piles, April & May, 2010						
Pile	Test	Approx. Depth ft	CAPWAP Computed Soil Resistance (a), kips			Estimated Ultimate Resistance (b) kips
			Total	Shaft	Toe	
1	End Drive	141	760	70	690	NA
1	Start 1st Restrike	141	930	330	600	NA
1	Start 2nd Restrike	142	900	660	240	~650+660=~1310
2	End Drive	142	600	40	560	NA
2	Start 1st Restrike	142	690	340	350	NA
2	Start 2nd Restrike	142	730	480	250	~540+480=~1020
3	End Drive	152	500	440	60	NA
3	Start 1st Restrike	152	620	530	90	NA
3	Start 2nd Restrike	152	650	560	90	650
4	End Drive	139	610	90	520	NA
4	Start 1st Restrike	139	1050	780	270	NA
4	Start 2nd Restrike	139	1000	850	150	~480+850=~1330
5	End Drive	140	500	170	330	NA
5	Start 1st Restrike	140	850	590	260	NA
5	Start 2nd Restrike	140	1000	770	230	~320+770=~1090
6	End Drive	118	500	70	430	NA
6	Start 1st Restrike	118	670	440	230	NA
6	Start 2nd Restrike	118	920	530	390	~400+530=~930
7	End Drive	136	320	100	220	NA
7	Start 1st Restrike	136	790	500	290	NA
7	Start 2nd Restrike	136	850	590	260	~850
8	End Drive	120	520	150	370	NA
8	Start 1st Restrike	120	710	340	370	~710
8	End 1st Restrike	126	600	150	450	NA
8	Start 2nd Restrike	126	880	600	280	~430+600=~1030

Notes:
(a) CAPWAP computed soil resistances are ultimate resistances for downward loads and they must be reduced by an appropriate Factor of Safety (ASD) or Resistance Factor (LRFD).
(b) For closed-end piles the Estimated Ultimate Resistance is based on synthesis of end bearing from driving with friction from restrike if the restrike penetration resistance suggests that the restrike did not fully mobilize the available restrike resistance.

Appendix B

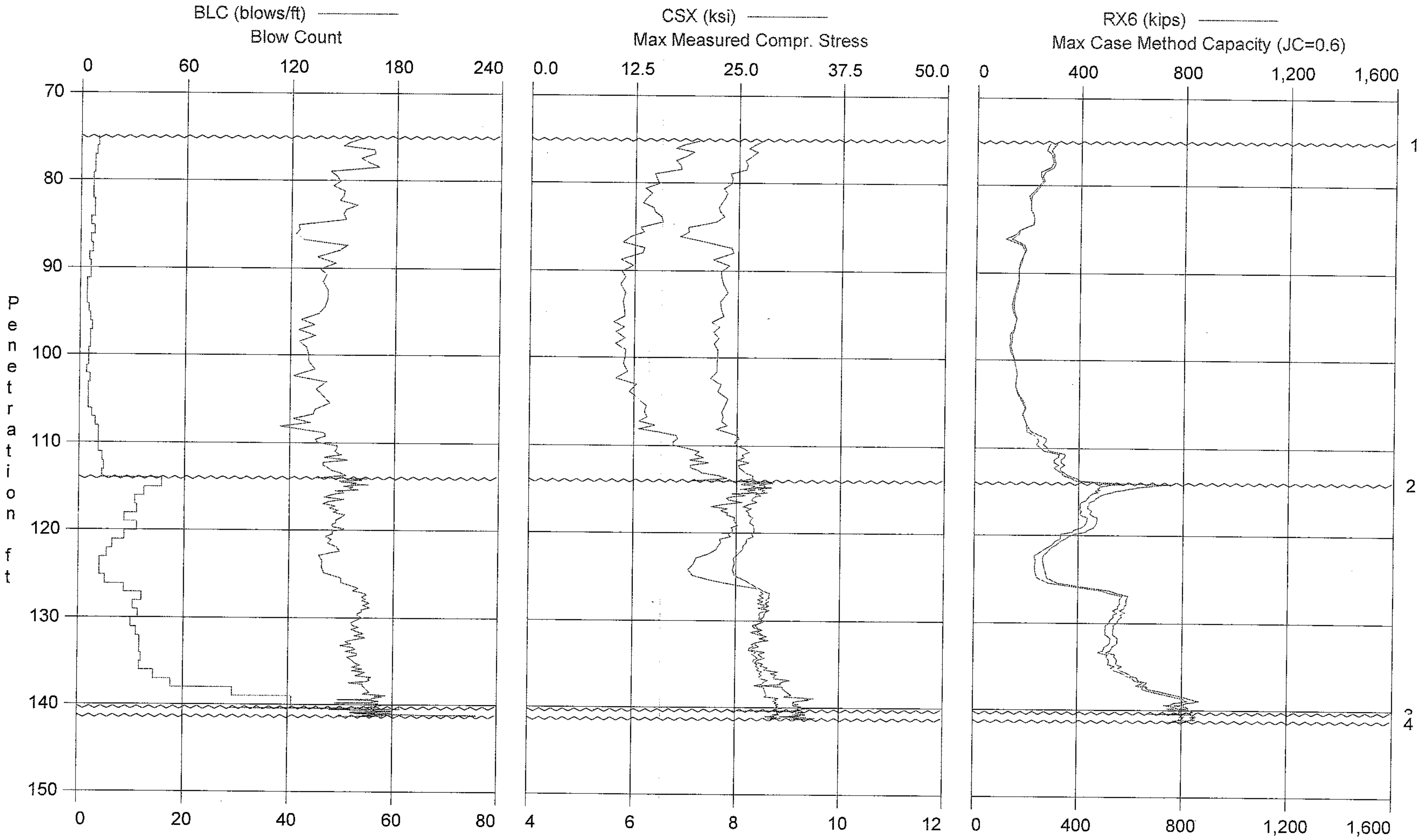
Summary of Case Method Field Results

PDIPLOT Ver. 2009.1 - Printed: 11-May-2010

Robert Miner Dynamic Testing, Inc. - Case Method Results

Test date: 14-Apr-2010

GCC, SR520, LOG YARD TEST PILES - PILE 1, D46-32PP24"x0.50" CLOSED END



1 - Start of test on 4/14/2010 at 8:21:44 AM
2 - Splice, then Restart after 6 hours
3 - Stop, mark inches.
4 - End of test on 4/15/2010 at 3:02:50 PM

Robert Miner Dynamic Testing, Inc.
Case Method Results

PDIPLOT Ver. 2009.1 - Printed: 11-May-2010

GCC, SR520, LOG YARD TEST PILES - PILE 1, D46-32
OP: RMDT:--RMINER

PP24"x0.50" CLOSED END
Test date: 14-Apr-2010

AR: 36.91 in^2
LE: 117.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.40

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
EMX: Max Transferred Energy
STK: O.E. Diesel Hammer Stroke
BPM: Blows per Minute

RP1: Case-Goble Capacity (JC=0.1)
RX4: Max Case Method Capacity (JC=0.4)
RX6: Max Case Method Capacity (JC=0.6)
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
10	76.00	10	AV10	27.3	27.9	52.011	7.05	44.4	464	350	296	271
			STD	0.7	0.8	1.741	0.19	0.6	49	9	12	12
			MAX	28.8	29.6	54.936	7.44	45.4	585	362	315	295
			@BL	2	2	2	2	8	1	1	3	1
19	77.00	9	AV9	26.6	27.5	53.867	6.98	44.6	370	345	292	284
			STD	0.7	0.8	3.275	0.21	0.6	19	14	11	13
			MAX	27.5	28.4	56.927	7.20	45.8	396	370	310	308
			@BL	16	16	14	16	11	11	16	16	16
27	78.00	8	AV8	25.9	26.9	54.199	6.83	45.1	319	337	293	285
			STD	0.5	0.5	2.326	0.12	0.4	8	10	10	11
			MAX	26.6	27.6	57.277	6.98	45.7	329	352	310	299
			@BL	20	27	20	20	22	21	23	23	23
35	79.00	8	AV8	25.0	26.6	52.425	6.65	45.7	279	289	263	250
			STD	1.4	1.1	6.065	0.36	1.2	23	17	20	18
			MAX	26.8	27.8	62.300	7.08	47.6	320	315	300	286
			@BL	29	29	29	29	35	28	28	28	28
42	80.00	7	AV7	24.4	26.5	49.176	6.48	46.2	260	266	252	247
			STD	0.5	1.0	1.332	0.15	0.5	15	7	8	8
			MAX	25.3	28.2	51.911	6.77	46.8	282	278	266	263
			@BL	36	36	36	36	39	36	38	38	38
49	81.00	7	AV7	23.6	25.8	48.836	6.31	46.8	242	251	230	220
			STD	0.7	0.9	2.608	0.20	0.7	19	12	11	11
			MAX	24.6	27.2	53.016	6.57	48.1	275	270	251	243
			@BL	49	44	44	44	46	49	43	43	43
56	82.00	7	AV7	23.3	24.5	49.797	6.22	47.1	273	221	206	202
			STD	0.3	0.8	1.137	0.07	0.3	23	10	4	7
			MAX	23.7	25.6	51.068	6.32	47.5	308	242	213	213
			@BL	54	51	51	54	55	56	52	54	54
64	83.00	8	AV8	22.7	25.7	50.853	6.25	47.0	291	232	207	207
			STD	0.8	2.1	1.991	0.17	0.6	53	16	9	8
			MAX	24.2	28.3	54.166	6.43	48.0	413	258	220	219
			@BL	59	62	63	64	58	62	63	59	63
72	84.00	8	AV8	23.0	29.3	50.140	6.44	46.4	230	245	219	219
			STD	0.9	1.4	2.470	0.22	0.7	41	9	5	5
			MAX	25.2	32.5	55.185	6.97	47.1	295	255	227	227
			@BL	71	71	71	71	65	67	71	71	71
78	85.00	6	AV6	21.2	29.9	47.722	6.43	46.4	209	239	204	203
			STD	1.6	0.9	4.571	0.22	0.8	17	16	28	28
			MAX	23.3	31.3	53.573	6.80	47.4	232	266	241	241
			@BL	73	75	74	75	77	74	75	74	74
86	86.00	8	AV8	18.6	28.4	40.675	6.04	47.8	197	194	152	143
			STD	0.7	1.0	1.361	0.18	0.7	18	12	15	22
			MAX	19.3	30.0	43.250	6.25	49.1	229	205	169	164
			@BL	83	79	83	83	86	81	83	84	82
92	87.00	6	AV6	20.2	30.2	42.393	6.23	47.2	233	205	167	151
			STD	3.4	2.8	9.855	0.56	1.9	62	39	34	48
			MAX	27.3	35.2	56.912	7.33	48.8	350	290	220	219
			@BL	92	92	92	92	90	91	91	91	91
99	88.00	7	AV7	24.5	34.4	50.782	6.21	47.2	247	220	185	182
			STD	1.0	1.4	2.866	0.21	0.8	25	13	12	13
			MAX	26.1	36.1	55.128	6.58	48.0	286	239	213	213
			@BL	94	94	97	97	96	93	96	93	93

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
104	89.00	5	AV5	22.2	31.2	44.695	5.75	48.9	222	205	176	172
			STD	0.8	0.9	2.165	0.19	0.8	4	12	13	17
			MAX	23.4	32.5	47.665	6.09	49.8	225	218	188	187
			@BL	102	102	104	102	103	100	100	103	100
110	90.00	6	AV6	23.5	34.0	47.465	5.88	48.4	221	192	161	159
			STD	0.8	0.8	1.746	0.14	0.6	16	12	9	10
			MAX	24.2	35.0	50.004	6.08	49.2	236	210	177	177
			@BL	105	105	105	105	110	110	106	106	106
116	91.00	6	AV6	23.1	34.3	46.459	5.81	48.7	214	173	168	166
			STD	0.8	1.0	1.952	0.15	0.6	6	6	8	10
			MAX	24.2	36.0	48.630	5.98	49.6	223	180	180	180
			@BL	116	116	115	116	113	111	116	116	116
120	92.00	4	AV4	23.2	34.1	46.129	5.73	49.0	208	166	160	156
			STD	0.6	0.8	0.690	0.10	0.4	19	12	13	16
			MAX	24.0	35.1	47.050	5.86	49.5	226	180	174	172
			@BL	120	120	117	120	118	119	118	119	119
124	93.00	4	AV4	23.8	34.4	47.083	5.82	48.6	205	161	150	146
			STD	0.5	1.5	0.667	0.11	0.4	14	4	8	10
			MAX	24.7	36.7	47.692	5.98	49.2	224	165	163	163
			@BL	122	122	121	122	121	122	122	122	122
128	94.00	4	AV4	22.9	33.4	47.022	5.78	48.8	197	159	145	138
			STD	0.9	2.0	1.957	0.12	0.5	15	5	6	7
			MAX	24.2	36.5	50.133	5.96	49.3	218	164	151	145
			@BL	126	126	126	126	125	126	125	126	126
133	95.00	5	AV5	23.3	34.9	46.702	5.87	48.5	208	160	150	146
			STD	0.8	1.6	1.924	0.17	0.7	27	5	7	10
			MAX	24.6	36.8	48.623	6.11	49.5	245	165	161	161
			@BL	133	133	129	133	131	129	129	133	133
139	96.00	6	AV6	22.3	33.7	43.078	5.65	49.3	184	156	155	154
			STD	0.7	1.0	1.776	0.12	0.5	17	5	6	7
			MAX	23.1	34.9	45.913	5.81	50.2	206	163	163	163
			@BL	136	135	135	136	139	136	137	137	137
146	97.00	7	AV7	22.4	34.9	43.098	5.72	49.1	194	151	145	141
			STD	0.6	1.0	2.105	0.14	0.6	20	5	7	9
			MAX	23.5	36.8	45.700	5.98	49.9	222	157	157	157
			@BL	143	143	143	143	145	143	143	143	143
152	98.00	6	AV6	22.6	35.4	44.166	5.78	48.8	222	149	138	132
			STD	0.5	1.0	1.543	0.15	0.6	8	11	12	12
			MAX	23.1	36.6	45.966	5.91	50.1	234	171	158	148
			@BL	152	152	152	148	147	152	149	149	149
158	99.00	6	AV6	22.5	35.7	42.413	5.75	48.9	212	143	135	132
			STD	0.6	1.4	2.348	0.20	0.8	9	7	6	7
			MAX	23.1	38.3	46.070	6.15	49.9	229	156	143	143
			@BL	157	157	157	157	155	154	155	158	158
163	100.00	5	AV5	22.6	35.5	42.957	5.80	48.7	230	160	145	141
			STD	0.5	0.9	0.935	0.13	0.5	1	9	10	12
			MAX	23.5	37.0	43.774	6.01	49.3	231	175	160	159
			@BL	163	163	163	163	159	163	159	162	162
168	101.00	5	AV5	22.4	35.0	43.472	5.77	48.9	216	153	151	151
			STD	0.5	0.9	1.471	0.10	0.4	7	4	5	5
			MAX	23.1	36.4	45.317	5.89	49.4	227	159	159	159
			@BL	166	165	168	165	167	165	166	166	166
172	102.00	4	AV4	22.5	34.8	44.677	5.87	48.4	223	162	160	160
			STD	0.4	0.7	1.651	0.16	0.6	7	4	5	5
			MAX	23.1	35.5	47.122	6.11	49.1	234	166	166	166
			@BL	171	171	171	171	169	170	171	171	171
178	103.00	6	AV6	22.0	34.9	42.546	5.74	49.0	210	161	153	151
			STD	0.6	0.8	2.972	0.19	0.7	24	15	15	15
			MAX	23.4	36.1	47.702	6.12	49.8	233	180	170	167
			@BL	178	176	178	178	175	175	175	175	175

GCC, SR520, LOG YARD TEST PILES - PILE 1, D46-32 PP24"x0.50" CLOSED END
OP: RMDT:--RMINER Test date: 14-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
183	104.00	5	AV5	22.9	35.1	45.369	5.98	48.0	221	164	162	161
			STD	0.8	1.8	2.958	0.22	0.9	15	6	8	11
			MAX	23.7	37.5	48.453	6.23	49.5	244	176	176	176
			@BL	180	179	183	179	182	183	183	183	183
188	105.00	5	AV5	23.7	34.7	46.628	6.08	47.7	238	190	182	177
			STD	0.6	1.1	1.598	0.13	0.5	12	4	5	5
			MAX	24.5	36.0	48.700	6.27	48.5	255	195	189	184
			@BL	185	186	185	185	188	185	186	186	186
193	106.00	5	AV5	23.5	34.8	47.424	6.27	47.0	246	210	194	191
			STD	0.3	0.5	1.414	0.09	0.3	14	7	7	8
			MAX	23.9	35.7	49.903	6.37	47.5	268	218	204	202
			@BL	190	193	190	193	189	189	191	191	191
200	107.00	7	AV7	23.2	33.1	44.284	6.21	47.2	234	198	185	184
			STD	0.6	1.2	1.317	0.16	0.6	33	4	8	10
			MAX	24.7	35.8	46.801	6.58	47.6	282	204	195	195
			@BL	200	200	200	200	198	196	195	200	200
209	108.00	9	AV9	23.4	34.0	41.948	6.24	47.1	283	259	200	197
			STD	0.8	1.9	2.415	0.22	0.8	38	24	9	7
			MAX	25.2	37.3	47.398	6.68	47.9	330	295	214	208
			@BL	207	207	207	207	205	207	208	208	204
220	109.00	11	AV11	23.9	35.5	42.622	6.50	46.2	362	304	246	221
			STD	1.3	2.2	3.978	0.34	1.1	41	25	24	20
			MAX	26.2	39.3	48.717	7.04	47.8	414	347	291	255
			@BL	219	217	219	217	210	217	218	220	220
231	110.00	11	AV11	24.9	36.7	45.636	6.80	45.2	387	331	270	242
			STD	0.5	0.8	1.769	0.14	0.4	18	15	13	11
			MAX	25.7	38.0	48.152	6.95	46.3	428	358	296	264
			@BL	224	228	223	228	225	221	223	223	230
242	111.00	11	AV11	26.1	39.1	48.794	7.13	44.2	404	373	317	294
			STD	0.8	1.0	2.016	0.21	0.6	28	28	35	34
			MAX	27.8	40.1	53.296	7.49	45.2	445	419	373	356
			@BL	238	239	238	238	232	239	239	239	239
255	112.00	13	AV13	25.7	40.3	48.667	7.26	43.8	429	389	337	310
			STD	0.7	0.9	2.088	0.18	0.5	17	14	12	15
			MAX	26.6	41.6	51.448	7.54	45.1	456	415	365	345
			@BL	255	247	255	247	250	247	255	255	255
269	113.00	14	AV14	25.6	40.1	47.059	7.20	44.0	422	393	336	320
			STD	0.7	1.2	2.204	0.21	0.6	25	7	17	19
			MAX	26.9	42.8	52.882	7.66	45.0	476	410	369	354
			@BL	256	256	256	256	266	267	264	269	268
282	114.00	13	AV13	26.8	42.0	49.243	7.67	42.6	495	430	390	373
			STD	0.7	1.0	3.586	0.27	0.7	34	19	24	23
			MAX	27.7	43.0	53.347	7.96	44.7	551	453	417	397
			@BL	275	271	281	281	270	281	277	282	280
329	115.00	47	AV47	28.3	36.3	51.778	8.33	41.0	1,096	813	624	493
			STD	1.4	2.2	3.655	0.25	0.6	66	66	69	35
			MAX	30.4	38.7	56.514	8.81	42.3	1,251	967	786	618
			@BL	286	287	319	284	326	286	286	284	284
366	116.00	37	AV34	27.9	35.4	50.562	8.13	41.5	980	675	485	444
			STD	0.8	1.1	2.238	0.27	0.7	38	34	21	13
			MAX	29.5	37.5	55.751	8.67	42.7	1,048	744	541	479
			@BL	341	333	341	341	366	330	330	330	330
398	117.00	32	AV16	26.7	33.7	47.819	7.83	42.2	875	566	438	407
			STD	0.8	1.2	2.210	0.26	0.7	32	30	10	6
			MAX	28.3	36.2	52.587	8.40	43.5	920	609	454	422
			@BL	380	380	380	380	396	380	370	368	368
430	118.00	33	AV16	26.5	33.0	48.501	7.78	42.3	805	503	451	413
			STD	0.6	0.8	1.621	0.20	0.5	28	16	16	17
			MAX	27.6	34.4	51.746	8.14	43.3	856	530	483	450
			@BL	418	414	418	418	416	402	402	412	412

GCC, SR520, LOG YARD TEST PILES - PILE 1, D46-32
PP24"x0.50" CLOSED END
OP: RMDT:--RMINER
Test date: 14-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
457	119.00	26	AV13	26.7	33.0	49.573	7.95	41.9	783	518	469	429
			STD	0.5	0.8	1.582	0.16	0.4	35	17	14	16
			MAX	27.6	33.9	52.013	8.29	42.7	830	549	485	448
			@BL	450	436	450	450	432	454	456	448	440
490	120.00	33	AV17	27.1	31.4	48.861	7.94	41.9	832	522	424	374
			STD	0.5	0.8	2.661	0.17	0.4	26	18	31	35
			MAX	27.7	33.1	57.250	8.20	42.8	864	544	468	418
			@BL	468	462	482	460	466	460	458	458	458
516	121.00	26	AV13	26.7	31.6	48.041	7.81	42.2	803	481	340	318
			STD	0.6	0.7	1.494	0.18	0.5	44	44	20	22
			MAX	27.6	32.5	50.106	8.05	43.2	862	536	381	354
			@BL	498	500	500	500	510	498	498	492	492
534	122.00	19	AV9	25.9	31.2	48.359	7.66	42.7	725	398	292	272
			STD	0.7	0.9	3.362	0.21	0.6	33	35	29	41
			MAX	26.8	32.4	54.233	7.95	43.6	801	477	317	307
			@BL	528	532	528	528	520	518	518	522	522
550	123.00	16	AV8	25.1	31.2	48.098	7.40	43.4	673	347	264	232
			STD	0.3	0.4	3.219	0.12	0.3	21	24	23	34
			MAX	25.5	31.9	54.641	7.58	43.8	708	384	300	280
			@BL	538	538	538	538	550	540	540	540	540
563	124.00	12	AV6	24.6	31.2	46.082	7.12	44.2	640	324	263	228
			STD	0.8	1.0	3.405	0.25	0.8	22	13	10	18
			MAX	25.9	32.5	49.285	7.50	45.4	672	345	273	245
			@BL	556	556	556	556	560	554	554	556	552
575	125.00	12	AV6	24.6	31.5	45.819	7.09	44.3	621	323	278	242
			STD	0.6	0.9	2.319	0.14	0.4	19	6	6	9
			MAX	25.3	32.7	48.572	7.32	44.9	638	333	285	252
			@BL	572	572	574	572	570	572	564	566	566
590	126.00	15	AV8	26.3	33.8	49.190	7.64	42.7	661	390	353	321
			STD	0.6	0.8	2.342	0.26	0.7	36	56	62	69
			MAX	27.3	35.1	53.473	8.11	43.5	728	505	464	436
			@BL	590	590	580	590	576	590	590	590	590
616	127.00	26	AV13	28.4	35.6	52.560	8.47	40.6	818	576	547	519
			STD	0.7	0.7	2.900	0.19	0.4	46	34	39	43
			MAX	29.7	36.6	56.255	8.83	41.4	899	652	640	628
			@BL	614	612	604	604	602	616	616	616	616
652	128.00	36	AV18	28.9	35.5	54.370	8.48	40.6	854	613	585	559
			STD	0.6	0.8	1.449	0.17	0.4	30	8	9	10
			MAX	30.0	36.9	56.546	8.79	41.4	923	625	600	576
			@BL	618	628	618	618	620	628	640	640	642
683	129.00	31	AV15	28.7	35.7	54.819	8.54	40.5	843	600	572	545
			STD	0.5	0.7	1.575	0.15	0.3	30	8	9	11
			MAX	29.7	36.9	58.912	8.77	41.1	916	616	591	567
			@BL	658	658	658	658	672	658	678	678	678
716	130.00	34	AV17	28.4	35.1	53.694	8.48	40.6	839	590	559	529
			STD	0.4	0.6	1.163	0.12	0.3	22	9	8	8
			MAX	29.2	35.9	55.603	8.65	41.2	894	605	573	541
			@BL	696	688	688	688	692	688	694	694	694
747	131.00	30	AV15	27.7	35.2	52.535	8.42	40.8	815	568	536	505
			STD	0.5	0.6	1.665	0.16	0.4	19	9	9	8
			MAX	28.5	36.2	55.061	8.64	41.6	869	583	550	520
			@BL	742	742	736	742	732	726	742	742	742
780	132.00	33	AV17	28.2	35.8	53.153	8.47	40.6	830	574	545	516
			STD	0.5	0.7	1.351	0.15	0.4	25	13	12	12
			MAX	29.4	37.2	55.799	8.74	41.7	898	596	565	536
			@BL	772	772	772	772	758	772	766	766	766
815	133.00	35	AV17	27.6	36.1	52.608	8.47	40.6	825	571	541	513
			STD	0.7	0.6	1.958	0.14	0.3	20	11	11	12
			MAX	28.7	37.1	55.575	8.68	41.3	886	590	560	533
			@BL	784	792	792	792	800	784	790	790	794

GCC, SR520, LOG YARD TEST PILES - PILE 1, D46-32
OP: RMDT:--RMINER

PP24"x0.50" CLOSED END
Test date: 14-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
850	134.00	35	AV18	27.0	35.6	51.467	8.41	40.8	800	562	531	501
			STD	0.5	0.8	1.838	0.15	0.4	18	16	18	20
			MAX	28.1	37.6	54.181	8.79	41.4	847	588	559	531
			@BL	840	840	840	840	824	840	840	850	850
886	135.00	36	AV18	27.4	36.0	52.598	8.49	40.6	804	582	551	521
			STD	0.4	0.7	1.576	0.12	0.3	13	14	15	16
			MAX	28.1	37.2	56.077	8.69	41.2	828	610	579	547
			@BL	858	858	858	858	880	852	884	884	884
921	136.00	35	AV17	27.8	35.8	52.979	8.62	40.3	802	600	575	551
			STD	0.6	0.8	1.967	0.21	0.5	19	18	20	23
			MAX	28.7	36.9	56.184	9.00	41.4	838	632	608	589
			@BL	914	918	914	914	906	918	912	912	918
964	137.00	43	AV22	28.2	36.7	53.995	8.82	39.8	800	654	636	619
			STD	0.6	0.8	1.540	0.20	0.4	19	24	24	26
			MAX	29.4	37.9	56.571	9.19	40.9	834	701	689	677
			@BL	954	944	954	954	922	930	956	956	956
1017	138.00	53	AV22	28.2	36.7	54.293	8.83	39.8	735	709	670	648
			STD	0.7	0.9	2.246	0.23	0.5	23	30	24	21
			MAX	29.2	38.3	59.402	9.20	41.4	763	773	717	675
			@BL	974	974	974	974	992	968	1006	1006	1002
1105	139.00	88	AV20	29.2	37.6	55.996	9.17	39.1	881	879	839	805
			STD	0.7	0.7	1.939	0.24	0.5	45	19	17	13
			MAX	30.6	39.2	59.862	9.66	39.9	961	912	867	823
			@BL	1104	1102	1102	1102	1082	1104	1102	1102	1084
1227	140.00	122	AV61	30.0	36.8	55.718	9.22	39.0	979	860	814	769
			STD	0.5	0.9	5.096	0.18	0.4	45	32	28	25
			MAX	31.6	39.2	61.512	9.78	39.7	1,052	911	865	819
			@BL	1212	1118	1212	1212	1196	1212	1106	1106	1106
1318	140.50	184	AV46	29.9	36.6	53.112	9.23	39.0	976	859	817	776
			STD	1.7	2.2	12.260	0.20	0.4	162	70	62	54
			MAX	31.3	38.7	59.145	9.68	39.7	1,068	898	854	812
			@BL	1316	1316	1316	1236	1280	1238	1310	1238	1238
1331	140.58	144	AV6	30.2	37.2	56.409	9.20	39.0	1,008	887	843	800
			STD	0.4	0.5	1.628	0.14	0.3	15	9	8	8
			MAX	30.7	37.6	57.941	9.40	39.4	1,029	900	855	810
			@BL	1326	1330	1326	1326	1324	1326	1320	1320	1320
1344	140.67	156	AV7	30.1	37.4	56.846	9.25	38.9	1,001	883	839	794
			STD	0.5	0.7	1.832	0.19	0.4	17	12	11	10
			MAX	30.8	38.2	58.739	9.48	39.7	1,019	897	852	807
			@BL	1336	1340	1336	1336	1334	1340	1336	1336	1336
1359	140.75	180	AV8	30.2	37.5	57.001	9.28	38.9	1,005	891	847	804
			STD	0.6	0.9	1.930	0.23	0.5	18	12	11	11
			MAX	31.2	39.1	59.931	9.66	39.5	1,023	915	869	824
			@BL	1348	1348	1350	1348	1359	1350	1350	1350	1350
1373	140.83	168	AV14	30.0	37.5	56.643	9.25	38.9	1,006	886	842	798
			STD	0.5	0.6	1.566	0.15	0.3	13	11	11	11
			MAX	30.9	38.7	59.605	9.54	39.5	1,025	903	859	815
			@BL	1365	1365	1365	1365	1361	1367	1364	1364	1368
1386	140.92	156	AV13	29.7	37.4	56.135	9.19	39.1	1,005	889	844	800
			STD	0.4	0.6	1.378	0.13	0.3	11	11	12	12
			MAX	30.4	38.3	58.235	9.40	39.4	1,022	912	870	828
			@BL	1377	1377	1380	1377	1386	1384	1376	1376	1376
1401	141.00	180	AV15	29.3	36.9	54.661	9.05	39.4	998	885	839	795
			STD	0.6	0.7	1.865	0.22	0.5	20	10	10	11
			MAX	30.7	38.7	59.033	9.61	39.9	1,048	909	865	823
			@BL	1392	1392	1392	1392	1399	1392	1391	1391	1391
1415	141.08	168	AV14	29.5	37.0	55.705	9.16	39.1	1,010	892	845	801
			STD	0.5	0.7	1.870	0.18	0.4	16	9	9	10
			MAX	30.6	38.6	59.070	9.54	39.8	1,046	908	863	819
			@BL	1413	1413	1413	1413	1402	1413	1412	1412	1412

GCC, SR520, LOG YARD TEST PILES - PILE 1, D46-32
OP: RMDT:--RMINER

PP24"x0.50" CLOSED END
Test date: 14-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
1431	141.17	192	AV16	29.7	36.2	55.786	9.21	39.0	1,014	872	824	778
			STD	0.7	1.0	2.226	0.22	0.5	22	12	12	13
			MAX	30.8	38.2	59.633	9.58	39.7	1,059	896	850	804
			@BL	1421	1416	1416	1421	1419	1421	1417	1417	1417
1450	141.25	228	AV19	29.7	35.8	56.247	9.26	38.9	1,022	860	814	771
			STD	0.8	1.0	2.772	0.28	0.6	27	13	14	15
			MAX	31.3	37.6	62.143	9.74	40.4	1,057	874	832	789
			@BL	1438	1438	1437	1437	1449	1443	1443	1443	1443
1468	141.33	225	AV18	29.9	35.5	56.095	9.30	38.9	1,024	862	818	776
			STD	0.6	0.7	2.536	0.21	0.4	21	14	14	14
			MAX	31.4	37.2	62.701	9.89	39.5	1,070	883	839	797
			@BL	1465	1465	1465	1465	1455	1465	1464	1464	1464

Time Summary

Drive	15 seconds	8:21:44 AM - 8:21:59 AM (4/14/2010) BN 1 - 12
Stop	1 day 1 second	8:21:59 AM - 8:22:00 AM
Drive	7 minutes 50 seconds	8:22:00 AM - 8:29:50 AM BN 13 - 282
Stop	6 hours 1 minute 46 seconds	8:29:50 AM - 2:31:36 PM
Drive	31 minutes 14 seconds	2:31:36 PM - 3:02:50 PM BN 283 - 1468

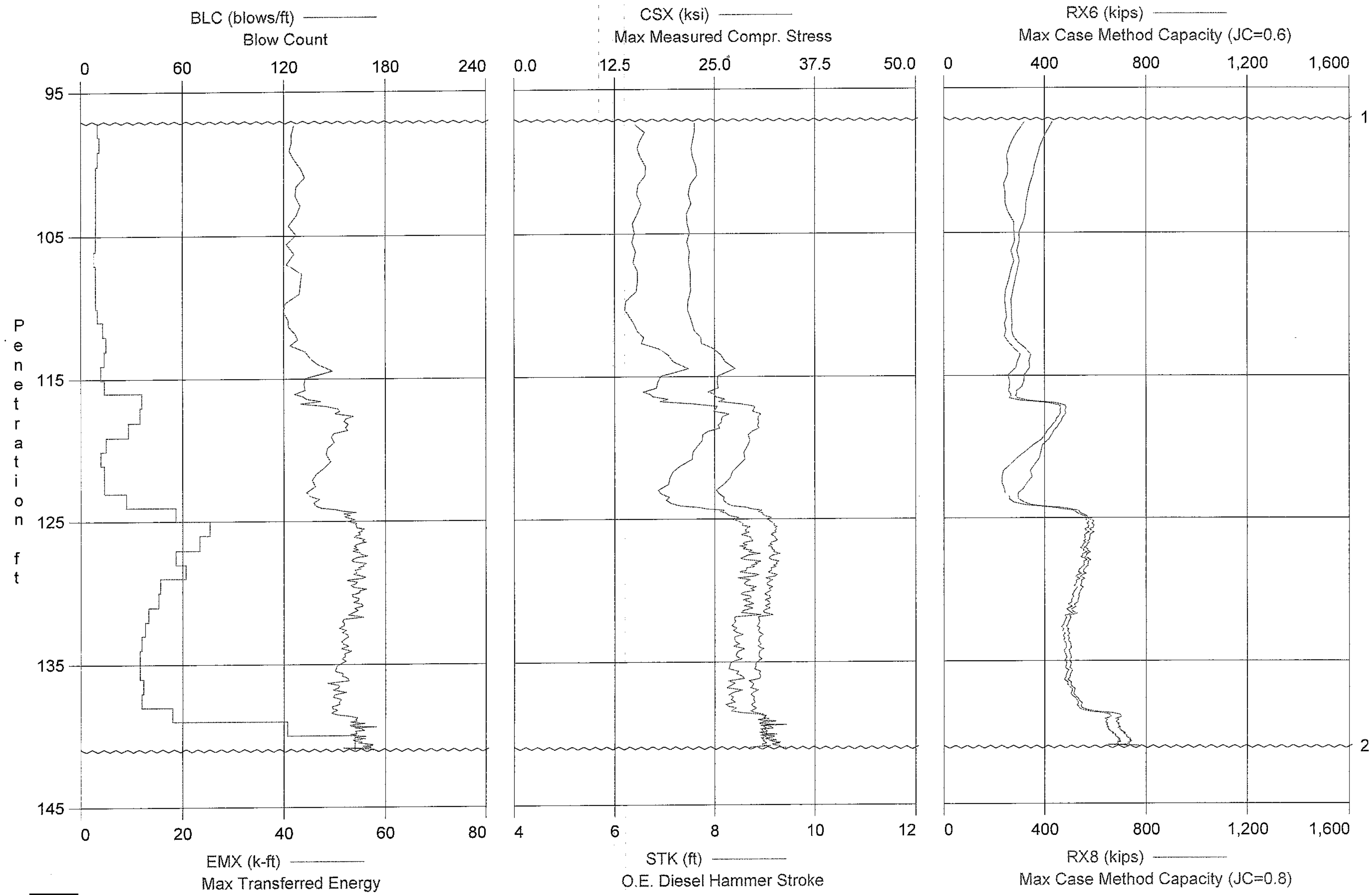
Total time [30:41:06] = (Driving [0:39:19] + Stop [30:01:47])

PDIPLOT Ver. 2009.1 - Printed: 11-May-2010

Robert Miner Dynamic Testing, Inc. - Case Method Results

Test date: 15-Apr-2010

GCC, SR520, LOG YARD TEST PILES - P2PP24"x0.401" CLOSED END



1 - Start of test on 4/15/2010 at 12:42:25 PM

2 - End of test on 4/15/2010 at 1:19:19 PM

Robert Miner Dynamic Testing, Inc.
Case Method Results

PDIPLOT Ver. 2009.1 - Printed: 11-May-2010

GCC, SR520, LOG YARD TEST PILES - P2
OP: RMDT:--RMINER

PP24"x0.401" CLOSED END
Test date: 15-Apr-2010

AR: 29.73 in^2
LE: 143.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.40

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
EMX: Max Transferred Energy
STK: O.E. Diesel Hammer Stroke
BPM: Blows per Minute

RP1: Case-Goble Capacity (JC=0.1)
RX4: Max Case Method Capacity (JC=0.4)
RX6: Max Case Method Capacity (JC=0.6)
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
10	98.00	10	AV10	22.3	24.4	41.386	6.73	45.5	713	540	425	312
			STD	1.8	2.4	3.133	0.45	1.3	45	26	16	16
			MAX	26.9	30.7	49.527	7.90	46.8	822	599	450	339
			@BL	2	2	2	2	7	2	2	2	1
21	99.00	11	AV11	22.4	23.2	41.526	6.53	46.1	691	510	390	271
			STD	0.7	0.6	1.601	0.18	0.6	18	12	9	9
			MAX	23.4	24.2	44.543	6.79	47.0	718	530	405	287
			@BL	18	18	18	18	14	12	12	12	14
31	100.00	10	AV10	22.3	23.5	42.281	6.53	46.0	679	497	375	256
			STD	0.7	0.6	1.531	0.14	0.5	15	10	8	10
			MAX	23.6	24.7	46.431	6.84	46.9	700	514	390	279
			@BL	31	31	31	31	23	31	24	24	31
40	101.00	9	AV9	22.6	23.9	43.097	6.59	45.9	671	483	358	253
			STD	0.6	0.6	1.998	0.15	0.5	17	9	7	11
			MAX	23.4	24.6	46.538	6.86	46.7	695	497	365	267
			@BL	32	32	32	32	36	32	32	35	32
49	102.00	9	AV9	22.1	23.7	42.920	6.50	46.1	653	468	344	244
			STD	0.6	0.6	1.401	0.12	0.4	15	10	9	12
			MAX	23.0	24.5	45.210	6.67	46.7	676	481	356	254
			@BL	42	48	41	42	46	42	42	44	42
58	103.00	9	AV9	21.8	23.4	42.559	6.48	46.2	638	453	329	242
			STD	0.5	0.5	1.251	0.12	0.4	11	4	6	7
			MAX	22.9	24.2	44.678	6.71	46.8	657	461	343	250
			@BL	56	56	55	56	50	56	54	50	54
67	104.00	9	AV9	21.6	23.5	42.749	6.48	46.2	626	445	324	259
			STD	0.6	0.6	1.604	0.15	0.5	15	8	7	16
			MAX	22.4	24.2	45.070	6.67	47.2	649	455	337	289
			@BL	59	65	59	65	63	59	59	61	67
76	105.00	9	AV9	21.7	23.1	41.271	6.37	46.6	616	431	309	278
			STD	0.4	0.5	1.427	0.09	0.3	11	12	15	5
			MAX	22.8	24.2	44.255	6.55	47.0	639	465	349	285
			@BL	76	68	76	76	69	68	68	68	75
85	106.00	9	AV9	21.8	22.8	41.655	6.40	46.5	609	419	300	280
			STD	0.7	0.4	3.418	0.16	0.6	14	12	13	16
			MAX	23.1	23.5	46.743	6.71	47.1	629	447	335	324
			@BL	85	85	85	85	80	85	81	81	81
93	107.00	8	AV8	21.7	22.8	40.461	6.38	46.6	609	420	301	279
			STD	0.7	0.4	2.042	0.17	0.6	20	20	31	37
			MAX	23.1	23.6	43.453	6.72	47.2	659	472	382	377
			@BL	93	93	86	93	89	93	93	93	93
102	108.00	9	AV9	21.9	22.8	42.902	6.42	46.4	603	408	289	265
			STD	0.5	0.4	1.720	0.14	0.5	12	8	8	7
			MAX	22.4	23.2	44.862	6.59	47.2	617	421	298	272
			@BL	99	94	98	98	96	94	102	94	102
111	109.00	9	AV9	22.1	22.8	43.460	6.46	46.3	604	406	281	255
			STD	0.9	0.8	2.397	0.19	0.6	20	11	7	9
			MAX	23.4	24.0	47.775	6.79	47.2	634	423	292	268
			@BL	110	108	108	108	111	110	110	103	103
120	110.00	9	AV9	21.7	22.5	40.946	6.29	46.9	594	399	270	243
			STD	0.5	0.4	1.330	0.14	0.5	10	9	7	5
			MAX	22.6	23.3	43.573	6.54	47.6	612	411	279	249
			@BL	116	116	113	113	117	113	113	112	120

GCC, SR520, LOG YARD TEST PILES - P2
OP: RMDT:--RMINER

PP24"x0.401" CLOSED END
Test date: 15-Apr-2010

BL#	depth	BLC	TYPE	CSX	CSI	EMX	STK	BPM	RP1	RX4	RX6	RX8
end	ft	bl/ft		ksi	ksi	k-ft	ft	**	kips	kips	kips	kips
130	111.00	10	AV10	21.8	22.7	40.104	6.24	47.1	593	398	271	250
			STD	0.6	0.7	1.575	0.15	0.5	16	11	9	10
			MAX	22.7	23.7	42.368	6.48	47.8	614	413	282	265
			@BL	128	122	130	128	121	122	122	128	128
143	112.00	13	AV13	22.3	23.6	41.473	6.41	46.4	602	402	270	244
			STD	0.6	0.8	1.518	0.14	0.5	14	9	6	6
			MAX	23.6	25.1	43.520	6.73	47.4	629	414	279	255
			@BL	138	138	138	138	134	138	138	133	142
158	113.00	15	AV15	23.5	25.5	42.403	6.59	45.9	636	427	288	251
			STD	1.0	1.2	2.055	0.21	0.7	25	18	15	11
			MAX	25.3	27.5	46.253	6.94	46.9	685	461	311	271
			@BL	158	158	158	158	153	158	158	158	156
172	114.00	14	AV14	25.7	28.8	45.060	7.07	44.3	708	484	340	298
			STD	0.6	0.7	1.449	0.13	0.4	19	14	11	11
			MAX	26.7	29.8	47.058	7.28	44.9	736	508	356	317
			@BL	169	169	169	169	163	166	166	166	166
184	115.00	12	AV12	26.7	29.8	47.612	7.26	43.8	725	489	333	276
			STD	1.0	1.1	2.308	0.26	0.8	27	18	13	17
			MAX	28.1	31.4	50.994	7.65	44.8	763	519	357	296
			@BL	176	177	176	177	184	176	175	175	173
198	116.00	14	AV14	25.4	28.4	43.924	6.86	45.0	688	464	315	261
			STD	0.6	0.7	1.436	0.14	0.4	14	10	8	6
			MAX	26.5	29.9	46.579	7.12	45.7	715	480	323	274
			@BL	194	194	188	194	198	194	194	194	194
234	117.00	36	AV36	25.7	28.9	44.826	6.94	44.8	685	454	322	295
			STD	1.5	1.5	2.601	0.36	1.1	39	27	59	59
			MAX	29.3	32.4	50.109	7.84	46.3	786	526	461	436
			@BL	232	232	232	232	201	234	234	233	234
269	118.00	35	AV35	30.2	32.5	51.492	8.10	41.5	834	572	480	459
			STD	0.8	0.9	2.098	0.21	0.5	21	15	8	8
			MAX	32.6	35.8	57.216	8.65	42.7	886	600	494	474
			@BL	244	244	244	244	241	244	244	243	243
297	119.00	28	AV28	30.2	31.2	51.658	8.03	41.7	835	578	451	427
			STD	0.8	0.9	2.257	0.22	0.6	18	11	14	15
			MAX	31.5	33.2	55.616	8.46	42.9	870	600	476	452
			@BL	272	272	271	272	294	272	284	271	270
312	120.00	15	AV15	29.2	29.9	49.149	7.70	42.5	809	561	409	378
			STD	0.5	0.6	1.293	0.12	0.3	14	11	17	20
			MAX	30.2	31.2	52.222	7.98	43.2	835	578	431	405
			@BL	306	306	306	306	309	306	301	299	299
324	121.00	12	AV12	28.7	29.2	48.724	7.56	42.9	792	547	383	307
			STD	0.6	0.6	1.347	0.16	0.4	16	12	9	21
			MAX	29.8	30.4	51.508	7.87	43.6	822	566	395	336
			@BL	319	319	319	319	316	319	319	314	315
338	122.00	14	AV14	27.4	27.8	46.818	7.20	44.0	749	510	351	247
			STD	0.8	0.8	2.013	0.20	0.6	20	14	11	18
			MAX	28.7	29.1	49.962	7.55	44.9	785	536	374	272
			@BL	325	325	330	325	337	325	328	328	327
352	123.00	14	AV14	26.2	26.5	45.792	7.07	44.3	720	488	334	235
			STD	0.9	0.8	1.837	0.18	0.5	26	19	15	7
			MAX	28.3	28.6	48.348	7.39	45.2	778	533	369	249
			@BL	339	339	339	339	352	339	339	339	351
379	124.00	27	AV27	26.0	26.8	45.833	7.04	44.4	681	442	303	261
			STD	0.7	0.7	1.634	0.17	0.5	13	11	13	17
			MAX	26.9	27.9	48.491	7.29	45.5	713	467	330	294
			@BL	378	378	365	378	353	360	360	378	379
435	125.00	56	AV56	30.4	31.2	52.023	8.08	41.6	775	534	503	486
			STD	1.4	1.2	2.302	0.35	0.9	32	42	70	75
			MAX	32.5	33.0	56.871	8.64	44.0	824	589	580	572
			@BL	430	430	400	430	384	430	432	432	432

GCC, SR520, LOG YARD TEST PILES - P2
OP: RMDT:--RMINER

PP24"x0.401" CLOSED END
Test date: 15-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
511	126.00	76	AV76	32.3	32.9	54.540	8.57	40.4	831	607	587	568
			STD	0.6	0.8	1.292	0.16	0.4	31	11	11	11
			MAX	33.8	35.0	58.344	9.02	41.4	895	632	610	589
			@BL	478	478	478	478	437	478	454	454	454
581	127.00	70	AV70	32.3	33.4	54.736	8.64	40.3	848	599	576	559
			STD	0.6	0.6	1.352	0.16	0.4	26	11	13	13
			MAX	33.6	34.9	58.153	9.03	41.1	897	626	609	591
			@BL	531	531	529	549	518	549	512	512	512
637	128.00	56	AV56	32.3	33.5	54.707	8.68	40.2	856	597	569	553
			STD	0.7	0.8	1.642	0.20	0.4	26	15	13	13
			MAX	33.9	35.3	59.102	9.24	41.2	914	641	606	591
			@BL	631	631	631	631	630	633	633	631	631
699	129.00	62	AV62	32.3	33.3	54.870	8.72	40.1	871	605	563	546
			STD	0.6	0.8	1.383	0.20	0.4	24	18	12	13
			MAX	33.9	35.4	58.460	9.18	41.0	939	653	588	573
			@BL	693	693	646	638	684	693	693	655	655
746	130.00	47	AV47	31.8	32.9	54.286	8.67	40.2	861	599	547	527
			STD	0.6	0.7	1.845	0.19	0.4	20	15	9	10
			MAX	33.2	34.6	59.098	9.21	41.3	905	634	571	554
			@BL	737	737	737	745	704	745	712	710	710
792	131.00	46	AV46	31.6	32.5	54.396	8.65	40.2	858	600	535	511
			STD	0.6	0.7	1.544	0.19	0.4	18	13	9	11
			MAX	32.8	34.1	57.404	9.09	41.1	913	639	553	531
			@BL	790	772	791	790	779	772	772	770	770
832	132.00	40	AV40	31.2	31.9	53.527	8.60	40.3	852	597	512	494
			STD	0.9	1.0	2.091	0.25	0.6	21	14	13	13
			MAX	33.0	33.9	58.685	9.10	41.2	898	626	533	518
			@BL	803	803	797	822	799	803	803	821	802
870	133.00	38	AV38	30.5	31.2	51.747	8.39	40.8	842	594	493	475
			STD	0.4	0.5	1.481	0.12	0.3	13	10	10	11
			MAX	31.6	32.7	55.212	8.69	41.4	862	610	514	496
			@BL	839	839	856	839	835	856	858	838	838
906	134.00	36	AV36	30.6	31.6	51.952	8.44	40.7	841	592	497	479
			STD	0.5	0.6	1.430	0.16	0.4	16	12	9	10
			MAX	32.0	33.3	55.305	8.85	41.4	872	617	516	499
			@BL	900	900	900	900	878	871	871	900	900
941	135.00	35	AV35	30.6	31.9	52.026	8.47	40.6	834	584	502	483
			STD	0.6	0.8	1.669	0.18	0.4	16	11	8	9
			MAX	31.7	33.1	55.044	8.79	41.3	868	610	518	503
			@BL	927	910	910	925	919	931	931	932	932
976	136.00	35	AV35	30.3	31.4	50.943	8.37	40.9	828	582	503	486
			STD	0.6	0.7	1.592	0.16	0.4	12	8	8	9
			MAX	31.3	32.7	53.712	8.63	41.8	857	603	518	504
			@BL	961	961	961	961	957	970	970	961	970
1013	137.00	37	AV37	29.9	31.1	50.933	8.40	40.8	816	572	504	491
			STD	0.7	0.8	1.963	0.20	0.5	18	14	10	12
			MAX	31.3	32.9	55.061	8.86	41.8	853	600	531	521
			@BL	1011	1011	1011	1011	989	977	988	1010	1010
1049	138.00	36	AV36	29.8	31.2	50.438	8.33	41.0	771	548	528	517
			STD	0.6	0.7	1.946	0.19	0.4	19	13	14	14
			MAX	31.1	32.9	54.484	8.75	41.9	810	579	557	544
			@BL	1039	1039	1039	1039	1024	1016	1048	1049	1046
1103	139.00	54	AV54	30.4	33.4	51.705	8.64	40.3	815	650	617	591
			STD	0.8	1.5	2.448	0.34	0.8	57	78	67	56
			MAX	32.0	36.4	56.612	9.41	41.8	922	752	709	668
			@BL	1099	1099	1099	1099	1052	1099	1099	1099	1099
1225	140.00	122	AV122	31.0	35.4	54.466	9.04	39.4	880	739	693	649
			STD	0.6	0.8	1.826	0.21	0.4	20	10	11	10
			MAX	33.1	37.7	60.791	9.77	40.6	946	766	722	679
			@BL	1151	1151	1151	1151	1106	1152	1224	1224	1224

GCC, SR520, LOG YARD TEST PILES - P2
OP: RMDT:--RMINER

PP24"x0.401" CLOSED END
Test date: 15-Apr-2010

BL#	depth	BLC	TYPE	CSX	CSI	EMX	STK	BPM	RP1	RX4	RX6	RX8
end	ft	bl/ft		ksi	ksi	k-ft	ft	**	kips	kips	kips	kips
1387	141.00	162	AV155	31.0	37.0	55.714	9.17	39.1	907	778	733	690
			STD	0.8	2.0	1.988	0.21	0.4	21	18	18	17
			MAX	33.8	44.1	60.420	9.89	40.2	974	840	793	746
			@BL	1234	1387	1381	1234	1344	1234	1371	1384	1384

Time Summary

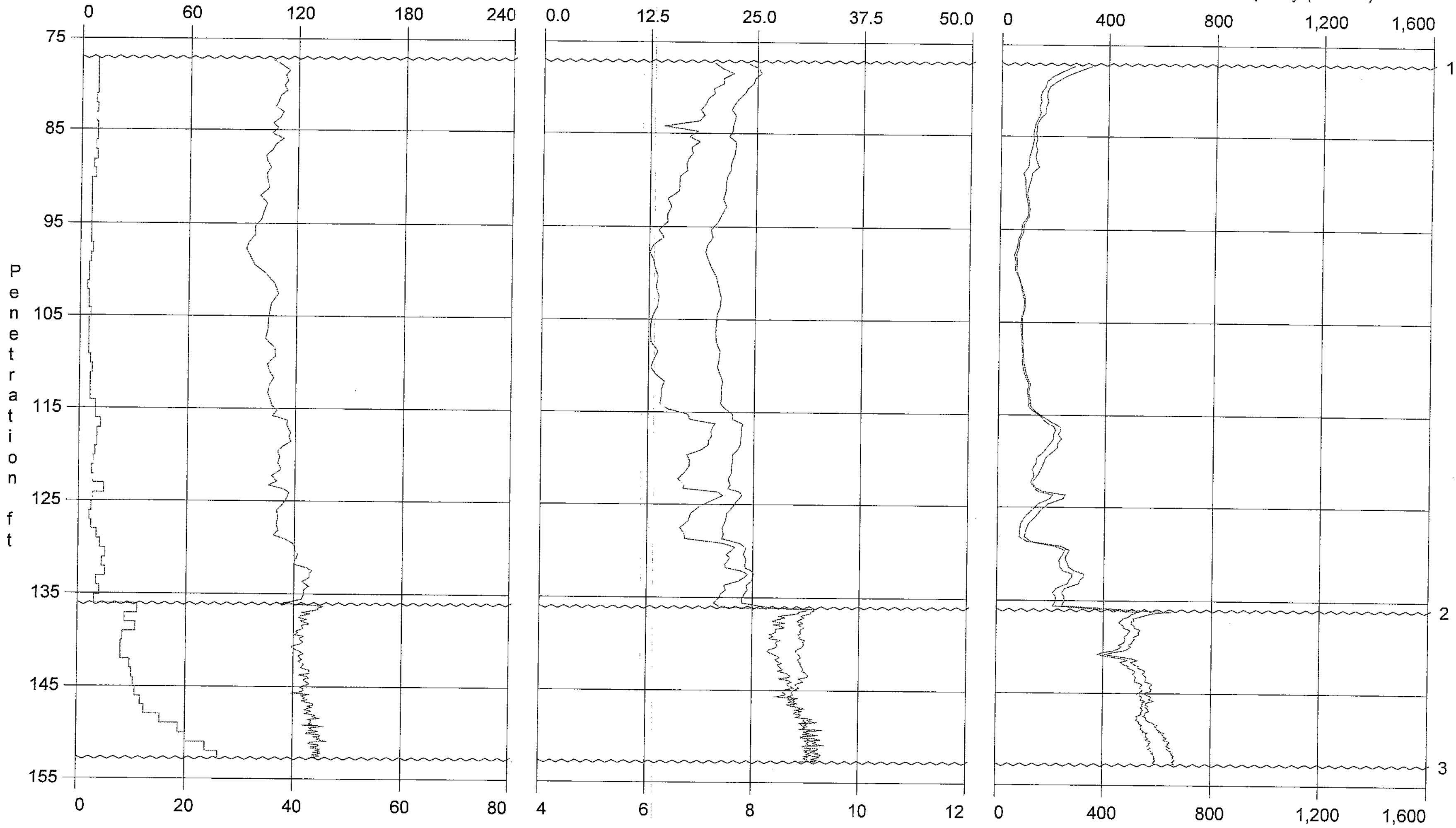
Drive 36 minutes 54 seconds 12:42:25 PM - 1:19:19 PM (4/15/2010) BN 1 - 1387

PDIPLOT Ver. 2009.1 - Printed: 11-May-2010

Robert Miner Dynamic Testing, Inc. - Case Method Results

Test date: 15-Apr-2010

GCC, SR520, LOG YARD TEST PILES - Pile 3, D46-32PP24"x0.401", OPEN END



1 - Start of test on 4/15/2010 at 9:23:49 AM
2 - Restart after 6 hours.
3 - End of test on 4/15/2010 at 3:58:58 PM

GCC, SR520, LOG YARD TEST PILES - Pile 3, D46-32
OP: RMDT:--RMINER

PP24"x0.401", OPEN END
Test date: 15-Apr-2010

AR: 29.73 in^2
LE: 138.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.40

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
EMX: Max Transferred Energy
STK: O.E. Diesel Hammer Stroke
BPM: Blows per Minute

RP1: Case-Goble Capacity (JC=0.1)
RX4: Max Case Method Capacity (JC=0.4)
RX6: Max Case Method Capacity (JC=0.6)
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
9	78.00	9	AV9	24.5	35.0	36.115	7.67	42.7	598	384	316	256
			STD	2.6	3.6	4.632	0.45	1.2	42	34	29	23
			MAX	26.2	37.8	40.872	8.59	44.5	686	468	386	308
			@BL	9	5	2	2	6	2	2	2	2
18	79.00	9	AV9	25.1	34.6	37.720	7.45	43.2	512	301	233	186
			STD	0.5	1.1	1.101	0.17	0.5	23	17	24	19
			MAX	26.0	35.9	39.137	7.75	44.2	543	329	272	217
			@BL	11	15	13	11	17	11	12	12	12
27	80.00	9	AV9	24.4	32.9	37.900	7.34	43.5	453	247	185	164
			STD	0.6	1.1	1.309	0.18	0.5	15	13	11	7
			MAX	25.6	34.9	40.585	7.70	44.0	482	270	207	176
			@BL	22	19	22	22	24	19	20	19	20
36	81.00	9	AV9	23.5	30.9	37.350	7.17	44.0	426	228	171	147
			STD	0.7	1.1	2.438	0.17	0.5	11	8	10	16
			MAX	24.3	32.4	42.360	7.44	44.6	440	238	181	162
			@BL	31	28	32	28	30	28	28	33	35
44	82.00	8	AV8	22.6	29.2	36.393	7.05	44.4	395	224	171	145
			STD	0.4	0.9	1.001	0.10	0.3	13	7	7	7
			MAX	23.4	30.3	38.383	7.29	44.7	412	234	178	151
			@BL	39	39	39	39	41	38	40	40	44
53	83.00	9	AV9	22.3	27.7	36.584	7.00	44.5	367	214	158	142
			STD	0.4	0.4	1.244	0.11	0.3	8	7	11	9
			MAX	22.9	28.7	38.265	7.17	45.0	376	224	172	158
			@BL	51	51	51	53	49	45	46	46	47
61	84.00	8	AV8	22.3	27.3	36.449	6.91	44.8	352	196	137	131
			STD	0.4	0.8	1.209	0.12	0.4	5	5	7	6
			MAX	23.1	28.7	37.972	7.06	45.5	360	206	150	145
			@BL	56	56	57	56	59	56	55	54	55
70	85.00	9	AV9	22.1	27.9	35.777	6.86	45.0	344	183	131	120
			STD	0.5	0.9	1.179	0.14	0.5	5	5	4	4
			MAX	22.9	29.0	37.063	7.05	45.8	353	191	139	130
			@BL	62	68	68	68	70	62	62	64	64
79	86.00	9	AV9	22.1	28.3	36.164	6.83	45.1	338	175	131	119
			STD	0.6	1.6	1.712	0.16	0.5	11	6	6	5
			MAX	23.1	30.2	37.968	7.03	46.0	352	188	140	129
			@BL	78	78	78	78	71	78	71	71	71
87	87.00	8	AV8	22.5	25.7	36.222	6.82	45.1	326	193	133	113
			STD	0.6	1.1	1.615	0.18	0.6	9	7	8	7
			MAX	23.6	27.8	38.770	7.13	46.2	341	205	146	129
			@BL	81	81	81	81	84	83	83	83	80
96	88.00	9	AV9	22.2	24.2	34.000	6.73	45.4	321	196	135	106
			STD	0.4	0.6	1.112	0.16	0.5	7	10	12	4
			MAX	22.9	25.1	35.580	6.99	46.2	332	213	153	115
			@BL	94	96	94	94	92	92	96	96	96
103	89.00	7	AV7	22.0	23.9	34.451	6.70	45.5	309	196	135	99
			STD	0.5	0.6	1.724	0.19	0.6	6	8	11	3
			MAX	23.1	25.1	37.913	7.13	46.0	321	202	144	103
			@BL	101	101	101	101	97	97	99	97	100
111	90.00	8	AV8	21.5	23.4	34.499	6.56	46.0	285	168	109	85
			STD	0.4	0.6	2.303	0.15	0.5	18	20	20	27
			MAX	22.4	24.6	39.121	6.86	46.5	308	196	132	107
			@BL	108	108	108	108	110	106	106	106	106

GCC, SR520, LOG YARD TEST PILES - Pile 3, D46-32
OP: RMDT:--RMINER

PP24"x0.401", OPEN END
Test date: 15-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
117	91.00	6	AV6	21.2	23.1	33.867	6.50	46.2	267	149	101	91
			STD	0.7	0.8	2.043	0.24	0.8	5	7	8	8
			MAX	22.0	23.9	36.098	6.78	47.8	277	162	111	98
			@BL	113	113	113	113	116	113	115	115	117
123	92.00	6	AV6	21.4	23.3	34.244	6.48	46.2	252	137	102	98
			STD	0.5	0.5	2.017	0.21	0.7	11	4	5	6
			MAX	22.0	23.8	37.864	6.78	47.1	270	145	111	109
			@BL	118	118	118	118	121	119	120	119	119
129	93.00	6	AV6	21.5	23.6	34.150	6.43	46.4	241	127	110	105
			STD	0.4	0.4	1.163	0.13	0.4	2	5	2	3
			MAX	22.1	24.3	35.714	6.62	47.0	244	135	113	108
			@BL	126	126	126	126	129	125	127	127	127
135	94.00	6	AV6	21.0	22.9	33.713	6.32	46.8	235	118	106	102
			STD	0.4	0.6	0.821	0.09	0.3	8	5	4	5
			MAX	21.8	24.0	34.962	6.49	47.1	248	125	111	108
			@BL	131	131	131	131	135	130	130	131	131
141	95.00	6	AV6	20.4	22.0	33.097	6.31	46.8	220	107	89	82
			STD	0.3	0.3	1.493	0.08	0.3	7	9	11	12
			MAX	20.7	22.5	36.186	6.40	47.3	229	122	105	100
			@BL	137	137	136	137	141	139	137	137	137
147	96.00	6	AV6	20.0	21.8	32.900	6.27	46.9	194	99	78	72
			STD	0.6	0.8	1.739	0.20	0.7	6	6	12	11
			MAX	20.7	22.8	36.059	6.54	48.3	202	105	89	84
			@BL	146	146	146	146	145	142	142	142	142
153	97.00	6	AV6	19.3	21.0	30.205	6.05	47.8	186	93	77	71
			STD	0.4	0.5	0.935	0.13	0.5	9	9	10	12
			MAX	19.9	21.8	31.577	6.25	48.7	198	105	88	86
			@BL	149	149	148	149	153	150	149	152	152
160	98.00	7	AV7	19.3	21.0	31.185	6.08	47.7	180	80	63	55
			STD	0.6	0.7	1.576	0.18	0.7	9	4	3	1
			MAX	20.0	21.9	33.293	6.31	48.6	197	85	66	57
			@BL	154	154	154	154	159	158	158	156	155
166	99.00	6	AV6	19.5	21.2	31.582	6.09	47.6	182	83	65	60
			STD	0.4	0.6	1.057	0.14	0.5	9	3	3	2
			MAX	19.9	21.7	33.261	6.26	48.6	201	86	70	64
			@BL	164	164	166	164	165	165	161	161	165
171	100.00	5	AV5	19.9	21.8	31.874	6.11	47.5	175	86	62	57
			STD	0.4	0.5	0.950	0.12	0.4	4	7	4	3
			MAX	20.3	22.5	33.379	6.27	48.3	179	95	65	61
			@BL	167	171	167	167	169	170	171	168	171
176	101.00	5	AV5	20.5	22.3	35.009	6.18	47.3	179	103	80	79
			STD	0.7	0.8	1.704	0.21	0.7	5	2	7	7
			MAX	21.4	23.6	37.239	6.50	48.3	186	105	89	87
			@BL	173	173	176	173	172	175	175	176	175
180	102.00	4	AV4	20.6	21.2	35.582	6.10	47.6	169	99	87	82
			STD	0.6	0.8	0.870	0.17	0.6	9	6	7	6
			MAX	21.7	22.3	36.925	6.38	48.3	178	107	98	92
			@BL	179	179	179	179	180	177	179	179	179
185	103.00	5	AV5	21.0	21.4	36.603	6.20	47.2	175	109	97	93
			STD	0.3	0.4	0.373	0.08	0.3	11	7	6	4
			MAX	21.4	21.9	37.006	6.31	47.7	194	117	105	100
			@BL	184	183	184	184	185	184	183	183	183
190	104.00	5	AV5	20.9	21.4	35.286	6.16	47.4	169	108	95	92
			STD	0.3	0.2	1.245	0.08	0.3	3	7	3	5
			MAX	21.2	21.6	36.535	6.26	47.9	172	119	98	96
			@BL	186	186	190	186	189	187	186	186	187
196	105.00	6	AV6	20.3	21.0	34.565	6.03	47.9	165	98	85	82
			STD	0.6	0.6	1.611	0.17	0.7	5	5	5	5
			MAX	21.1	21.8	36.651	6.23	48.9	172	106	95	90
			@BL	194	194	194	194	196	191	193	193	193

GCC, SR520, LOG YARD TEST PILES - Pile 3, D46-32 PP24"x0.401", OPEN END
OP: RMDT:--RMINER Test date: 15-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
201	106.00	5	AV5	20.6	21.1	35.201	6.07	47.7	155	98	85	82
			STD	0.4	0.4	1.183	0.14	0.5	3	5	4	3
			MAX	21.4	21.9	37.197	6.31	48.2	159	104	90	87
			@BL	197	197	197	197	198	197	199	200	201
206	107.00	5	AV5	20.6	20.9	34.803	6.09	47.6	166	99	86	83
			STD	0.6	0.6	2.420	0.20	0.8	8	4	3	6
			MAX	21.2	21.6	38.416	6.31	49.0	176	105	91	91
			@BL	202	202	202	202	204	206	206	205	205
211	108.00	5	AV5	20.4	20.8	34.429	6.03	47.8	175	103	91	86
			STD	0.6	0.6	1.777	0.16	0.6	8	3	2	3
			MAX	21.0	21.3	36.957	6.22	48.9	189	106	94	90
			@BL	211	211	211	211	209	211	211	209	210
216	109.00	5	AV5	21.0	21.6	36.102	6.17	47.3	189	104	90	88
			STD	0.4	0.3	1.565	0.12	0.4	8	5	7	6
			MAX	21.3	22.0	37.275	6.35	48.0	202	108	98	93
			@BL	212	216	216	212	215	216	215	215	213
222	110.00	6	AV6	20.7	21.6	35.738	6.08	47.7	203	105	93	87
			STD	0.4	0.5	1.132	0.11	0.4	7	4	5	5
			MAX	21.4	22.5	37.279	6.23	48.4	212	112	102	96
			@BL	218	218	218	218	219	218	219	219	219
229	111.00	7	AV7	20.7	21.8	34.476	6.07	47.7	203	111	103	96
			STD	0.4	0.6	1.046	0.11	0.4	15	6	8	8
			MAX	21.5	23.0	36.398	6.28	48.4	222	122	115	108
			@BL	227	227	227	227	224	229	225	225	225
235	112.00	6	AV6	21.2	22.7	35.733	6.29	46.9	239	123	117	113
			STD	0.3	0.4	1.184	0.12	0.4	7	2	3	5
			MAX	21.9	23.5	37.490	6.54	47.3	249	126	122	120
			@BL	235	235	234	235	231	233	233	233	233
241	113.00	6	AV6	21.1	22.3	35.168	6.26	47.0	237	119	111	106
			STD	0.5	0.6	0.992	0.13	0.5	10	5	5	5
			MAX	21.6	23.2	36.259	6.41	47.8	249	124	117	113
			@BL	237	237	239	237	236	239	236	236	236
247	114.00	6	AV6	21.0	22.1	34.605	6.20	47.2	236	126	119	114
			STD	0.5	0.6	0.847	0.13	0.5	12	6	8	11
			MAX	21.6	22.7	35.866	6.37	47.9	248	135	129	125
			@BL	244	244	247	244	246	244	247	247	247
256	115.00	9	AV9	21.6	23.3	35.697	6.39	46.5	242	147	136	130
			STD	0.7	1.1	2.111	0.25	0.9	19	20	17	20
			MAX	22.6	24.9	39.085	6.79	48.0	274	189	165	160
			@BL	256	256	251	256	250	256	256	255	256
265	116.00	9	AV9	22.4	24.6	36.009	6.77	45.3	322	224	187	170
			STD	0.3	0.3	1.145	0.12	0.4	20	20	22	19
			MAX	22.9	25.2	38.458	6.92	45.9	359	251	230	209
			@BL	258	258	258	258	262	265	265	265	265
277	117.00	12	AV12	23.6	25.6	38.672	7.26	43.8	366	280	233	212
			STD	0.4	0.5	1.072	0.13	0.4	5	10	8	11
			MAX	24.2	26.4	40.467	7.46	44.3	377	299	249	227
			@BL	276	266	276	276	269	275	270	270	275
287	118.00	10	AV10	23.4	25.3	38.546	7.15	44.1	355	280	231	205
			STD	0.3	0.4	1.007	0.10	0.3	9	7	7	9
			MAX	23.7	25.9	40.035	7.30	44.6	364	289	243	220
			@BL	282	282	284	284	280	282	282	285	285
297	119.00	10	AV10	23.4	25.4	39.085	7.14	44.1	318	268	218	183
			STD	0.2	0.3	0.847	0.08	0.2	13	14	12	12
			MAX	23.8	26.0	40.338	7.29	44.4	333	289	240	203
			@BL	290	290	290	290	289	288	291	291	288
306	120.00	9	AV9	22.6	24.7	36.823	6.82	45.1	276	225	185	151
			STD	0.4	0.5	1.335	0.17	0.5	17	9	11	12
			MAX	23.6	25.6	38.632	7.18	45.9	313	243	202	171
			@BL	298	298	298	298	304	299	298	300	300

GCC, SR520, LOG YARD TEST PILES - Pile 3, D46-32
OP: RMDT:--RMINER

PP24"x0.401", OPEN END
Test date: 15-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
314	121.00	8	AV8	22.5	24.7	37.117	6.80	45.2	255	210	166	134
			STD	0.3	0.3	1.235	0.10	0.3	5	10	12	12
			MAX	22.9	25.2	39.174	6.98	45.7	263	226	191	155
			@BL	307	307	312	312	308	307	308	308	308
321	122.00	7	AV7	22.2	24.6	36.617	6.64	45.7	242	168	145	133
			STD	0.5	0.6	2.138	0.18	0.6	6	10	6	5
			MAX	23.1	25.6	40.458	6.92	46.5	248	188	153	139
			@BL	317	317	317	317	321	321	315	318	318
329	123.00	8	AV8	22.1	24.4	35.988	6.63	45.7	260	159	132	128
			STD	0.4	0.5	1.146	0.12	0.4	7	4	8	8
			MAX	22.9	25.6	38.252	6.89	46.2	269	167	150	146
			@BL	323	323	323	323	324	329	324	329	329
343	124.00	14	AV14	22.9	25.3	37.048	7.09	44.3	337	255	214	180
			STD	0.8	0.9	1.661	0.36	1.1	48	60	50	35
			MAX	23.9	26.4	39.468	7.52	46.2	402	329	287	246
			@BL	338	342	342	342	332	339	338	338	338
351	125.00	8	AV8	23.1	25.6	38.500	7.20	43.9	327	248	196	156
			STD	0.4	0.4	0.891	0.13	0.4	18	27	27	19
			MAX	23.8	26.5	40.242	7.48	44.6	356	287	239	190
			@BL	345	345	349	345	351	345	345	345	345
358	126.00	7	AV7	22.3	24.7	37.440	6.94	44.7	292	205	157	121
			STD	0.4	0.5	1.452	0.13	0.4	5	10	10	13
			MAX	22.9	25.4	39.502	7.11	45.3	298	225	170	144
			@BL	354	354	356	354	358	352	352	352	352
364	127.00	6	AV6	21.8	24.2	36.788	6.79	45.2	275	171	124	91
			STD	0.2	0.6	0.735	0.08	0.3	7	12	10	7
			MAX	22.0	24.9	37.539	6.90	45.6	283	187	140	99
			@BL	364	364	364	362	359	360	359	359	359
371	128.00	7	AV7	21.4	24.1	36.588	6.63	45.7	239	144	106	83
			STD	0.5	0.6	1.458	0.16	0.5	10	8	6	8
			MAX	22.1	24.8	38.730	6.83	46.8	252	153	112	95
			@BL	367	367	370	367	369	365	366	367	369
381	129.00	10	AV10	21.5	24.3	36.510	6.75	45.3	254	157	123	105
			STD	0.5	0.7	1.222	0.16	0.5	38	39	37	33
			MAX	22.6	25.8	38.029	7.14	46.0	347	244	204	171
			@BL	381	381	372	381	374	381	381	381	381
393	130.00	12	AV12	24.0	27.0	39.623	7.61	42.8	409	310	261	243
			STD	0.5	0.5	1.403	0.16	0.4	15	13	13	17
			MAX	24.9	28.2	42.200	7.94	43.5	424	326	276	263
			@BL	388	388	393	388	383	387	390	390	390
408	131.00	15	AV15	24.0	26.4	40.134	7.53	43.0	400	306	259	236
			STD	0.4	0.6	1.227	0.14	0.4	5	9	7	5
			MAX	24.7	27.3	41.777	7.80	43.6	408	318	270	244
			@BL	402	402	405	402	396	399	394	405	394
421	132.00	13	AV13	24.1	27.2	40.036	7.49	43.1	398	325	280	244
			STD	0.3	0.4	0.595	0.10	0.3	15	21	21	17
			MAX	24.5	27.8	41.129	7.65	43.9	425	378	338	297
			@BL	414	421	414	414	412	419	421	421	421
436	133.00	15	AV15	25.0	27.7	42.859	7.85	42.2	408	362	317	275
			STD	0.4	0.6	1.011	0.15	0.4	14	7	8	7
			MAX	25.9	28.8	45.229	8.21	42.6	432	373	331	290
			@BL	426	426	426	426	432	423	426	426	426
446	134.00	10	AV10	24.5	26.6	42.026	7.56	42.9	359	315	273	237
			STD	0.5	0.8	0.913	0.19	0.5	23	31	29	25
			MAX	25.3	28.0	43.657	7.88	43.8	396	371	328	284
			@BL	438	438	440	438	444	439	438	438	438
458	135.00	12	AV12	24.1	25.3	42.127	7.43	43.3	325	284	249	216
			STD	0.6	0.7	1.215	0.17	0.5	9	7	8	9
			MAX	25.1	26.7	44.225	7.75	44.1	339	295	260	229
			@BL	449	449	449	449	454	447	451	455	454

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PP24"x0.401", OPEN END
Test date: 15-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
467	136.00	9	AV9	23.9	24.8	40.651	7.31	43.6	310	282	250	218
			STD	0.7	0.7	2.573	0.18	0.5	7	12	13	13
			MAX	24.9	25.7	43.129	7.56	44.6	326	298	267	236
			@BL	467	467	461	461	465	467	467	467	467
500	137.00	33	AV33	31.4	35.3	43.255	8.84	39.8	935	684	575	507
			STD	1.9	2.5	3.877	0.23	0.5	86	94	66	34
			MAX	33.2	37.0	46.464	9.25	40.9	1,140	924	780	636
			@BL	471	489	476	471	500	469	469	469	469
526	138.00	26	AV26	30.6	35.1	41.739	8.49	40.6	815	567	511	467
			STD	0.5	0.7	1.440	0.17	0.4	10	8	9	9
			MAX	31.7	36.4	44.186	8.80	41.6	839	581	527	486
			@BL	502	502	503	502	513	503	526	526	526
558	139.00	32	AV32	30.7	34.6	41.676	8.46	40.7	800	574	527	489
			STD	0.4	0.6	1.044	0.14	0.3	18	10	8	8
			MAX	31.4	35.8	43.688	8.74	41.4	827	591	542	505
			@BL	527	531	530	527	555	530	530	531	531
583	140.00	25	AV25	30.8	34.4	41.043	8.42	40.7	758	551	506	472
			STD	0.6	0.6	1.279	0.17	0.4	10	7	8	10
			MAX	32.1	35.9	43.817	8.85	41.6	778	564	521	492
			@BL	576	576	576	576	568	563	566	562	559
607	141.00	24	AV24	30.5	34.2	40.615	8.37	40.9	706	504	455	419
			STD	0.5	0.6	1.267	0.14	0.3	32	32	37	35
			MAX	31.3	36.0	42.946	8.59	41.6	758	554	508	473
			@BL	589	606	589	606	596	584	584	586	586
631	142.00	24	AV24	30.5	34.6	41.196	8.45	40.7	731	548	505	467
			STD	0.5	0.6	1.239	0.15	0.4	19	17	21	22
			MAX	31.7	36.1	43.550	8.83	41.2	758	570	531	494
			@BL	625	625	619	625	610	619	619	618	618
660	143.00	29	AV29	30.8	35.0	41.478	8.52	40.5	755	582	544	507
			STD	0.5	0.6	1.281	0.15	0.3	9	13	14	14
			MAX	32.0	36.3	44.082	8.86	41.3	770	607	573	538
			@BL	640	640	640	640	639	640	651	651	651
690	144.00	30	AV30	31.2	35.3	42.447	8.64	40.2	752	600	560	521
			STD	0.6	0.7	1.164	0.14	0.3	5	10	11	11
			MAX	32.5	36.8	44.392	8.94	41.1	767	622	585	548
			@BL	671	671	671	671	682	668	690	690	690
721	145.00	31	AV31	30.4	34.4	42.378	8.71	40.1	755	610	574	541
			STD	0.5	0.6	0.904	0.12	0.3	7	11	11	8
			MAX	31.4	35.5	44.517	8.97	40.6	770	628	591	554
			@BL	692	714	714	714	705	693	709	709	693
753	146.00	32	AV32	29.8	33.7	41.614	8.62	40.3	731	607	573	542
			STD	0.6	0.8	1.425	0.19	0.4	17	13	14	13
			MAX	31.3	35.6	45.031	9.13	41.2	772	639	604	574
			@BL	751	751	751	751	745	749	749	749	749
788	147.00	35	AV35	30.1	35.1	42.479	8.78	39.9	774	599	565	545
			STD	0.6	0.8	1.482	0.20	0.4	10	14	15	9
			MAX	31.6	36.7	46.067	9.24	40.8	801	636	599	562
			@BL	783	787	783	787	755	783	783	783	783
825	148.00	37	AV37	30.6	35.3	42.762	8.86	39.8	818	615	559	533
			STD	0.5	0.6	1.216	0.16	0.3	18	12	8	11
			MAX	31.9	36.8	45.716	9.26	40.3	849	639	575	561
			@BL	815	815	815	815	791	825	820	825	796
871	149.00	46	AV46	31.1	35.9	43.681	9.06	39.3	888	662	592	540
			STD	0.5	0.7	1.270	0.17	0.4	20	14	13	8
			MAX	32.3	37.3	46.696	9.50	40.1	914	682	609	554
			@BL	835	835	835	835	851	848	848	871	834
927	150.00	56	AV56	31.3	35.4	43.645	9.07	39.3	929	692	624	560
			STD	0.6	0.7	1.518	0.19	0.4	14	12	11	9
			MAX	32.9	37.0	47.654	9.54	40.0	957	715	645	575
			@BL	892	892	892	892	875	917	917	917	921

GCC, SR520, LOG YARD TEST PILES - Pile 3, D46-32
OP: RMDT:--RMINER

PP24"x0.401", OPEN END
Test date: 15-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
987	151.00	60	AV60	31.5	35.9	44.585	9.17	39.1	957	719	642	573
			STD	0.6	0.8	1.544	0.18	0.4	11	7	9	8
			MAX	32.8	37.5	48.230	9.54	40.1	977	736	660	586
			@BL	977	981	977	977	931	984	981	981	946
1058	152.00	71	AV71	31.5	36.1	44.065	9.17	39.1	980	746	656	585
			STD	0.6	0.7	1.311	0.16	0.3	13	12	10	8
			MAX	33.2	38.3	47.864	9.63	39.8	1,006	766	673	600
			@BL	1015	1015	1015	1015	999	1051	1051	1050	1050
1116	152.74	78	AV58	31.5	35.5	44.362	9.20	39.0	990	761	663	593
			STD	0.6	0.7	1.297	0.15	0.3	9	6	7	6
			MAX	33.1	37.1	47.772	9.66	39.7	1,013	772	682	607
			@BL	1063	1063	1063	1063	1065	1063	1082	1075	1075

Time Summary

Drive	10 minutes 25 seconds	9:23:49 AM - 9:34:14 AM (4/15/2010) BN 1 - 467
Stop	6 hours 5 minutes 59 seconds	9:34:14 AM - 3:40:13 PM
Drive	18 minutes 45 seconds	3:40:13 PM - 3:58:58 PM BN 468 - 1120

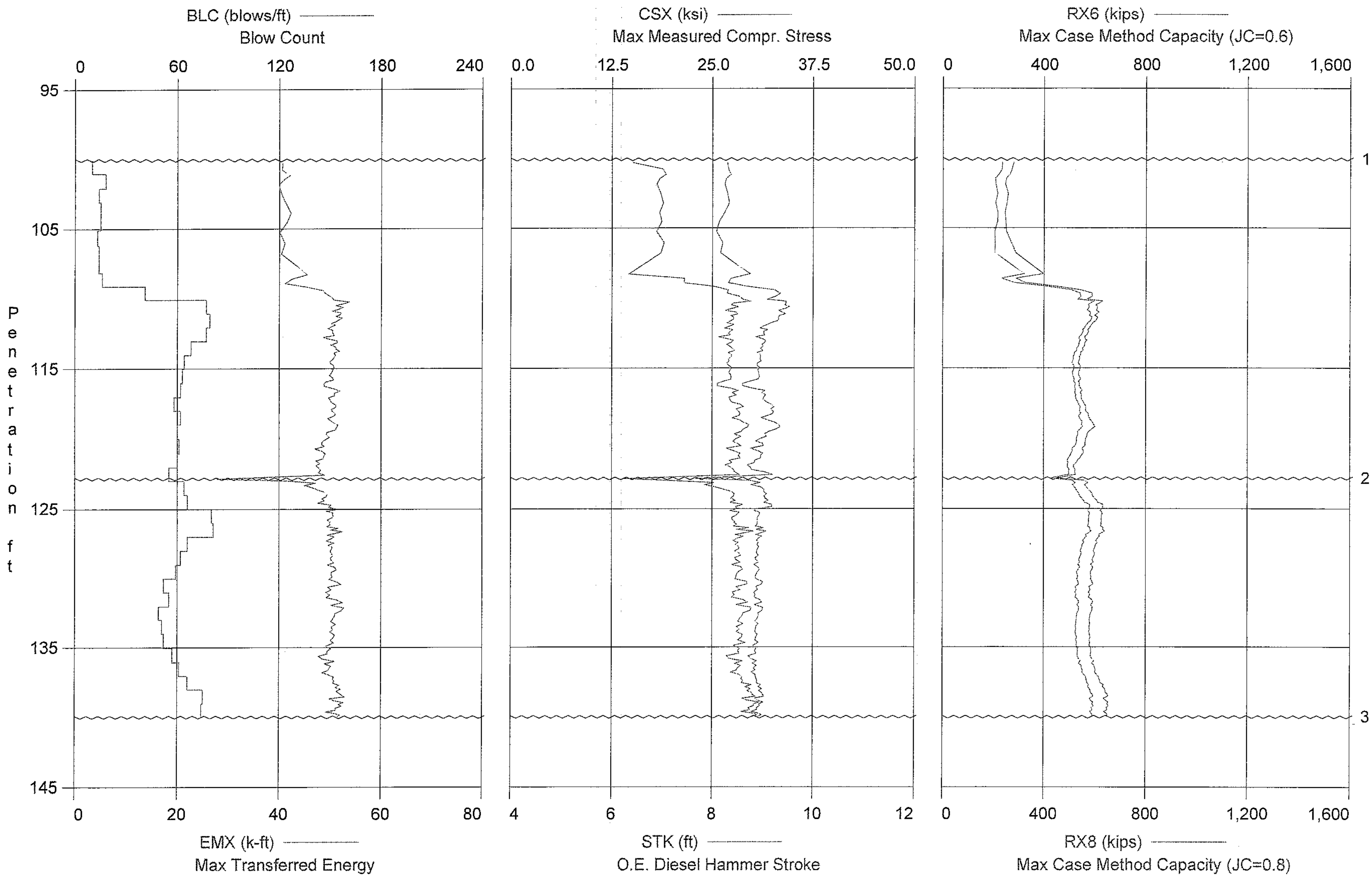
Total time [6:35:09] = (Driving [0:29:10] + Stop [6:05:59])

PDIPLOT Ver. 2009.1 - Printed: 11-May-2010

Robert Miner Dynamic Testing, Inc. - Case Method Results

Test date: 22-Apr-2010

KIEWIT GENERAL, CASTING YARD - PILE 4 PP24x0.401", D46-32



1 - Start of test on 4/22/2010 at 8:35 AM
2 - Pause, refuel D46-32.
3 - End of test on 4/22/2010 at 10:00 AM

Robert Miner Dynamic Testing, Inc.
Case Method Results

PDIPLOT Ver. 2009.1 - Printed: 11-May-2010

KIEWIT GENERAL, CASTING YARD - PILE 4
OP: RMDT:--RMINER

PP24x0.401", D46-32
Test date: 22-Apr-2010

AR: 29.73 in^2
LE: 146.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.40

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
EMX: Max Transferred Energy
STK: O.E. Diesel Hammer Stroke
BPM: Blows per Minute

RP1: Case-Goble Capacity (JC=0.1)
RX4: Max Case Method Capacity (JC=0.4)
RX6: Max Case Method Capacity (JC=0.6)
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
10	101.00	10	AV10	26.9	29.1	40.550	7.01	44.5	675	430	277	235
			STD	0.5	0.6	1.075	0.14	0.4	21	20	13	11
			MAX	28.0	30.1	42.521	7.27	44.9	713	464	298	259
			@BL	10	10	10	1	4	1	1	1	1
27	102.00	18	AV14	26.9	28.8	41.232	7.00	44.6	654	403	254	213
			STD	0.4	0.5	1.139	0.10	0.3	21	20	13	12
			MAX	27.5	29.6	42.670	7.14	45.2	684	432	283	241
			@BL	12	12	13	12	16	12	12	12	12
42	103.00	14	AV7	26.9	28.3	40.317	6.97	44.6	653	406	262	216
			STD	0.8	0.8	1.995	0.18	0.6	32	30	12	7
			MAX	28.1	29.5	43.031	7.26	45.6	699	452	287	233
			@BL	41	41	31	41	29	41	41	41	31
57	104.00	15	AV8	26.8	27.6	41.898	6.99	44.6	644	398	246	213
			STD	0.6	0.7	2.257	0.14	0.4	19	19	9	7
			MAX	28.1	28.8	46.931	7.27	45.1	678	426	261	227
			@BL	55	43	55	55	49	43	43	55	55
71	105.00	15	AV7	25.6	27.2	40.947	6.92	44.8	591	370	247	215
			STD	0.6	0.7	1.407	0.17	0.5	11	10	6	6
			MAX	26.5	28.1	43.128	7.16	45.6	605	384	254	222
			@BL	63	63	63	63	59	63	59	69	69
85	106.00	13	AV7	25.9	26.6	40.677	6.98	44.6	605	392	271	207
			STD	0.9	0.6	1.805	0.18	0.6	34	44	34	6
			MAX	27.1	27.5	43.541	7.23	45.4	649	444	311	220
			@BL	81	77	77	77	73	77	77	83	81
99	107.00	14	AV7	25.9	26.5	40.395	6.97	44.6	607	399	274	205
			STD	0.5	0.6	1.329	0.10	0.3	29	43	40	7
			MAX	27.0	27.7	43.187	7.17	44.9	654	463	336	214
			@BL	93	93	93	93	91	99	99	99	93
131	109.00	16	AV16	29.6	33.1	176.793	7.32	43.6	634	467	339	283
			STD	12.1	14.8	527.813	0.29	0.8	169	86	117	130
			MAX	76.1	89.6	2,220.964	7.75	45.3	742	782	774	766
			@BL	107	107	107	131	101	131	107	107	107
172	110.00	41	AV20	32.3	37.6	48.755	8.34	40.9	866	620	551	511
			STD	1.7	2.1	2.947	0.33	0.8	49	50	52	48
			MAX	34.7	40.5	52.918	8.93	42.6	948	696	610	558
			@BL	157	165	165	165	133	157	157	161	165
249	111.00	77	AV39	33.7	36.3	51.863	8.47	40.6	895	663	609	575
			STD	1.1	1.6	1.696	0.22	0.5	24	21	22	22
			MAX	35.3	40.8	55.944	9.13	41.9	935	718	674	637
			@BL	185	175	185	185	233	185	187	187	187
328	112.00	79	AV39	32.8	34.9	51.184	8.39	40.8	876	646	600	578
			STD	1.1	1.4	1.676	0.18	0.4	25	16	12	11
			MAX	34.9	37.5	54.657	8.80	42.0	928	679	623	599
			@BL	253	257	251	253	255	251	251	251	251
405	113.00	77	AV39	31.2	33.1	50.145	8.31	41.0	847	619	571	550
			STD	0.6	0.8	1.263	0.16	0.4	18	8	10	10
			MAX	32.3	34.5	52.458	8.55	41.9	880	634	590	570
			@BL	345	341	401	337	381	401	337	337	337
473	114.00	68	AV34	31.2	32.9	51.086	8.37	40.9	862	614	556	532
			STD	0.7	0.7	1.275	0.15	0.4	15	10	9	8
			MAX	32.4	34.2	54.117	8.77	41.6	902	638	573	549
			@BL	453	419	453	453	411	453	453	409	425

KIEWIT GENERAL, CASTING YARD - PILE 4
OP: RMDT:--RMINER
PP24x0.401", D46-32
Test date: 22-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
537	115.00	64	AV32	30.8	32.4	50.471	8.33	41.0	849	598	540	516
			STD	0.5	0.6	1.113	0.14	0.3	15	11	7	6
			MAX	31.9	33.7	53.111	8.68	41.6	884	625	554	529
			@BL	483	483	483	483	493	483	483	537	525
600	116.00	63	AV31	30.6	32.5	50.172	8.34	40.9	831	580	540	516
			STD	0.7	0.9	1.255	0.16	0.4	19	12	7	6
			MAX	31.9	34.5	52.006	8.65	41.7	865	606	549	526
			@BL	569	569	549	569	567	539	539	545	545
662	117.00	62	AV31	30.3	32.5	50.256	8.32	41.0	817	584	544	523
			STD	1.3	1.9	1.576	0.21	0.5	42	16	7	5
			MAX	32.8	35.9	53.896	8.80	42.1	885	613	563	540
			@BL	655	655	655	655	605	655	655	655	655
720	118.00	58	AV29	32.0	35.9	50.468	8.47	40.6	854	599	558	535
			STD	0.7	1.1	1.402	0.18	0.4	15	9	10	10
			MAX	33.2	37.5	53.088	8.83	41.8	888	618	580	558
			@BL	703	715	703	667	677	667	667	703	703
782	119.00	62	AV31	32.1	36.9	50.476	8.53	40.5	850	626	578	544
			STD	0.9	1.1	1.139	0.17	0.4	17	11	10	7
			MAX	33.6	38.7	52.586	8.87	41.4	881	646	597	556
			@BL	729	729	731	767	755	769	779	777	731
842	120.00	60	AV30	32.4	37.0	50.302	8.60	40.3	860	635	583	540
			STD	1.0	1.4	1.562	0.17	0.4	20	14	14	10
			MAX	34.2	39.6	53.727	8.99	41.0	896	664	611	561
			@BL	787	787	787	787	815	787	793	793	793
903	121.00	61	AV31	30.8	34.3	48.406	8.45	40.7	839	608	560	522
			STD	0.9	1.2	1.526	0.19	0.4	19	10	8	9
			MAX	32.1	36.4	51.426	8.76	41.7	871	623	571	536
			@BL	869	869	899	903	853	903	847	849	859
963	122.00	60	AV30	30.4	33.3	48.022	8.41	40.8	830	590	535	497
			STD	0.9	1.4	1.365	0.17	0.4	16	11	14	8
			MAX	32.1	36.1	50.580	8.71	41.6	867	618	565	517
			@BL	905	905	905	921	931	905	905	905	905
1018	123.00	55	AV26	29.7	32.4	44.220	8.11	41.6	818	581	513	486
			STD	3.0	3.5	7.286	0.70	2.0	69	45	31	30
			MAX	34.0	38.0	51.055	8.79	47.4	938	708	606	555
			@BL	993	1017	973	973	1009	1017	1017	1017	1017
1082	124.00	64	AV32	31.0	34.0	47.070	8.12	41.5	903	665	574	525
			STD	1.0	1.1	2.285	0.28	0.7	20	13	11	10
			MAX	33.2	36.7	51.328	8.65	42.9	951	692	595	543
			@BL	1063	1027	1063	1063	1021	1063	1063	1081	1081
1148	125.00	66	AV33	31.8	35.1	49.125	8.45	40.7	908	673	613	562
			STD	1.0	2.0	1.744	0.19	0.4	33	15	18	17
			MAX	34.1	39.6	52.029	8.82	41.5	959	701	642	589
			@BL	1121	1137	1125	1121	1115	1133	1133	1137	1133
1228	126.00	80	AV40	30.6	31.3	50.227	8.45	40.7	894	677	627	578
			STD	0.4	0.5	1.228	0.13	0.3	35	6	6	6
			MAX	31.5	33.1	54.134	8.82	41.5	932	689	639	590
			@BL	1175	1149	1149	1149	1211	1187	1149	1175	1175
1309	127.00	81	AV41	30.9	31.8	50.523	8.58	40.4	921	682	628	577
			STD	0.7	0.7	1.673	0.21	0.5	20	12	10	9
			MAX	32.4	33.2	54.705	9.02	41.6	967	709	646	594
			@BL	1279	1279	1279	1279	1233	1279	1279	1279	1279
1375	128.00	66	AV33	30.7	31.7	50.150	8.49	40.6	909	666	606	554
			STD	0.5	0.5	1.280	0.14	0.3	12	8	9	9
			MAX	31.9	33.0	52.195	8.80	41.3	938	685	622	571
			@BL	1323	1323	1323	1323	1327	1323	1323	1311	1311
1437	129.00	62	AV31	30.5	31.5	50.166	8.49	40.6	892	653	594	542
			STD	0.4	0.4	1.125	0.12	0.3	13	7	7	7
			MAX	31.4	32.3	53.538	8.76	41.1	916	666	604	552
			@BL	1429	1429	1429	1429	1435	1429	1387	1429	1389

KIEWIT GENERAL, CASTING YARD - PILE 4
OP: RMDT:--RMINER

PP24x0.401", D46-32
Test date: 22-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
1495	130.00	59	AV29	30.6	31.6	50.488	8.52	40.5	890	648	588	535
			STD	0.5	0.4	1.214	0.13	0.3	14	8	7	7
			MAX	31.6	32.3	53.123	8.77	41.1	915	665	605	552
			@BL	1463	1463	1459	1459	1439	1463	1445	1459	1445
1548	131.00	52	AV26	30.8	32.1	50.605	8.57	40.4	888	646	586	532
			STD	0.5	0.5	1.407	0.16	0.4	12	10	8	7
			MAX	32.0	33.1	54.041	8.98	41.1	922	667	599	543
			@BL	1519	1519	1519	1519	1541	1519	1513	1519	1519
1603	132.00	55	AV28	30.5	32.0	50.433	8.51	40.5	880	640	584	531
			STD	0.6	0.6	1.590	0.16	0.4	15	8	8	7
			MAX	32.1	33.6	54.764	9.03	41.4	919	660	603	546
			@BL	1585	1585	1585	1585	1569	1585	1585	1585	1585
1652	133.00	49	AV24	30.9	32.6	51.510	8.66	40.2	887	642	587	534
			STD	0.6	0.7	1.887	0.19	0.4	16	10	8	8
			MAX	31.9	33.7	54.967	8.98	40.9	912	656	599	546
			@BL	1611	1611	1609	1609	1605	1649	1609	1621	1609
1703	134.00	51	AV26	30.4	32.3	50.392	8.53	40.5	878	634	580	527
			STD	0.4	0.4	0.937	0.12	0.3	11	4	4	4
			MAX	31.2	33.1	52.690	8.75	41.4	899	641	586	535
			@BL	1675	1669	1675	1669	1681	1675	1675	1659	1659
1755	135.00	52	AV26	30.4	32.5	50.259	8.56	40.4	879	635	581	529
			STD	0.6	0.7	1.686	0.19	0.4	15	8	6	6
			MAX	31.9	34.3	53.691	8.94	41.5	918	654	596	544
			@BL	1735	1735	1735	1721	1751	1735	1721	1721	1721
1812	136.00	57	AV28	30.1	32.2	49.292	8.46	40.7	872	641	588	536
			STD	0.7	0.7	1.828	0.20	0.5	15	9	8	7
			MAX	31.9	34.1	54.263	9.00	41.5	906	668	612	557
			@BL	1811	1811	1811	1811	1791	1757	1811	1811	1811
1873	137.00	61	AV31	30.2	32.8	49.558	8.50	40.6	856	652	601	550
			STD	0.6	0.6	1.364	0.16	0.4	17	8	7	8
			MAX	31.4	33.9	52.513	8.83	41.5	887	669	616	564
			@BL	1873	1843	1813	1873	1861	1821	1873	1873	1853
1939	138.00	66	AV33	30.6	33.5	50.935	8.64	40.3	836	683	625	572
			STD	0.6	0.6	1.275	0.16	0.4	23	12	10	7
			MAX	32.4	35.0	54.480	9.13	40.8	871	711	649	588
			@BL	1925	1925	1925	1925	1881	1901	1925	1925	1925
2014	139.00	75	AV37	31.0	32.9	51.764	8.78	39.9	815	706	646	588
			STD	0.7	1.0	1.831	0.21	0.5	16	12	11	9
			MAX	32.4	34.8	55.896	9.34	41.0	857	732	670	608
			@BL	2011	1943	2011	2011	1989	1943	2011	2011	2011
2088	140.00	74	AV44	30.7	31.7	51.279	8.79	39.9	826	706	648	590
			STD	0.6	0.6	1.569	0.18	0.4	21	11	9	8
			MAX	31.9	32.9	55.079	9.18	40.8	879	729	667	607
			@BL	2088	2088	2088	2088	2055	2083	2088	2088	2033

Time Summary

Drive	2 minutes 25 seconds	8:35:51 AM - 8:38:16 AM (4/22/2010) BN 1 - 107
Stop	9 minutes 37 seconds	8:38:16 AM - 8:47:53 AM
Drive	22 minutes 6 seconds	8:47:53 AM - 9:09:59 AM BN 109 - 1011
Stop	24 minutes 29 seconds	9:09:59 AM - 9:34:28 AM
Drive	26 minutes 31 seconds	9:34:28 AM - 10:00:59 AM BN 1015 - 2088

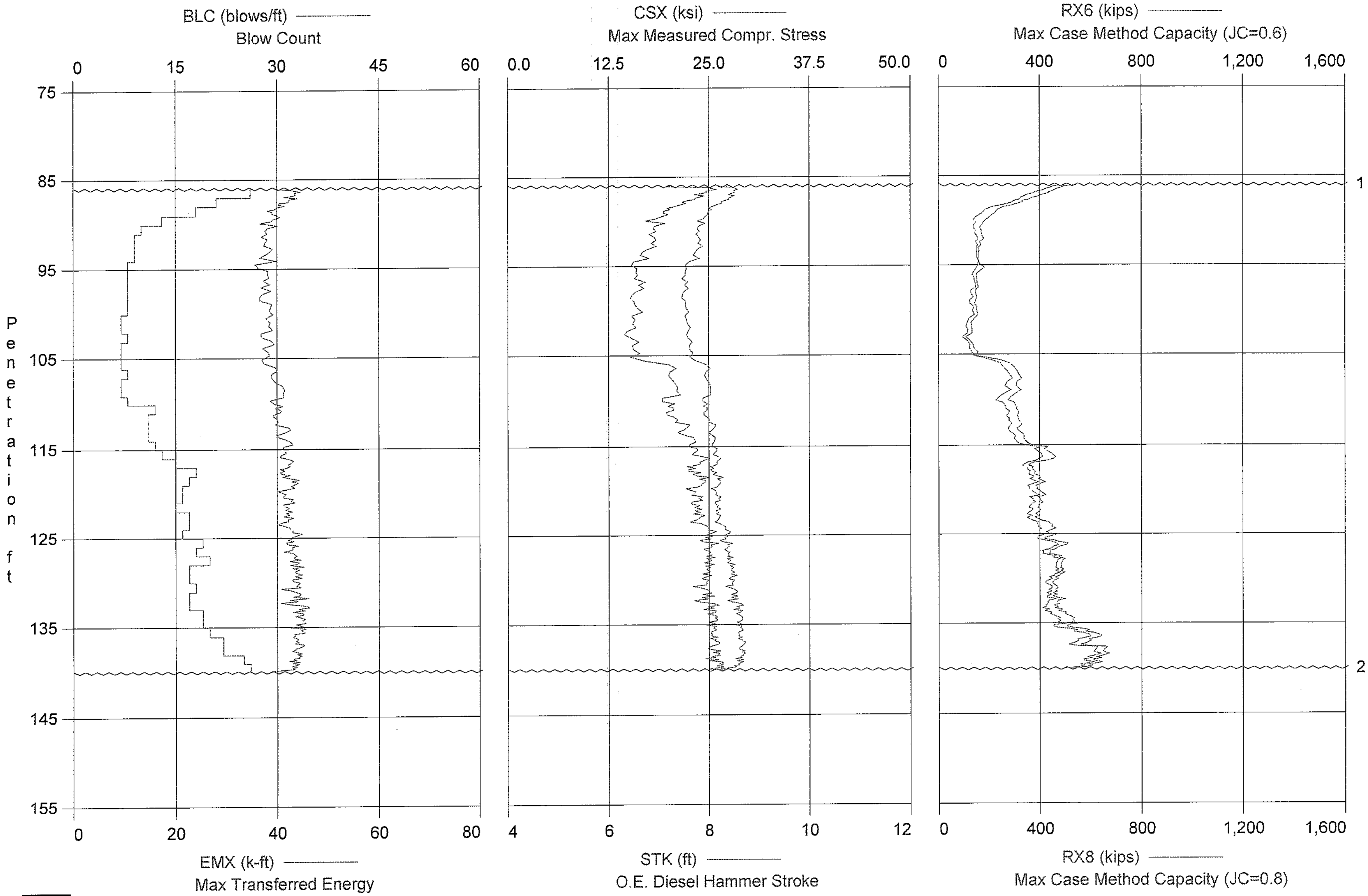
Total time [1:25:08] = (Driving [0:51:02] + Stop [0:34:06])

PDIPLOT Ver. 2009.1 - Printed: 11-May-2010

Robert Miner Dynamic Testing, Inc. - Case Method Results

Test date: 21-Apr-2010

KIEWIT GENERAL, CASTING YARD - PILE 5 PP24x0.401", D46-32



1 - Start of test on 4/21/2010 at 4:04 PM

2 - End of test on 4/21/2010 at 4:22 PM

Robert Miner Dynamic Testing, Inc.
Case Method Results

PDIPLOT Ver. 2009.1 - Printed: 11-May-2010

KIEWIT GENERAL, CASTING YARD - PILE 5
OP: RMDT:--RMINER

PP24x0.401", D46-32
Test date: 21-Apr-2010

AR: 29.73 in^2
LE: 142.50 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.40

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
EMX: Max Transferred Energy
STK: O.E. Diesel Hammer Stroke
BPM: Blows per Minute

RP1: Case-Goble Capacity (JC=0.1)
RX4: Max Case Method Capacity (JC=0.4)
RX6: Max Case Method Capacity (JC=0.6)
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
26	87.00	26	AV26	27.9	30.5	42.864	8.03	41.7	734	539	473	424
			STD	1.7	1.8	3.330	0.34	0.8	62	50	35	34
			MAX	30.5	31.9	49.134	9.43	42.9	951	698	592	516
			@BL	2	8	2	2	25	2	2	1	1
47	88.00	21	AV21	27.3	30.6	41.856	7.69	42.6	603	443	374	326
			STD	0.6	0.5	1.337	0.17	0.5	37	32	30	22
			MAX	28.5	31.6	44.321	7.99	43.3	658	495	432	373
			@BL	28	28	28	28	43	28	27	27	27
65	89.00	18	AV18	25.3	27.6	39.270	7.24	43.8	476	327	266	221
			STD	0.7	1.0	1.406	0.19	0.5	31	29	32	36
			MAX	27.3	30.3	42.750	7.74	44.8	524	374	321	284
			@BL	49	49	49	49	64	49	50	50	50
78	90.00	13	AV13	24.4	26.3	38.988	7.00	44.6	391	261	206	153
			STD	0.8	1.1	1.768	0.22	0.7	28	14	14	14
			MAX	25.8	28.0	41.512	7.29	45.8	446	284	231	179
			@BL	66	67	66	67	78	66	69	67	67
88	91.00	10	AV10	24.0	25.7	38.667	6.99	44.6	346	223	170	141
			STD	0.5	0.6	1.239	0.15	0.5	9	10	8	7
			MAX	25.0	26.6	41.067	7.29	45.2	364	242	185	154
			@BL	81	81	81	81	86	81	79	79	79
97	92.00	9	AV9	23.7	26.1	37.756	6.92	44.8	342	221	173	148
			STD	0.5	0.8	1.181	0.18	0.5	35	7	11	9
			MAX	24.9	27.9	39.719	7.26	45.5	439	236	201	167
			@BL	97	97	97	97	90	97	97	97	97
106	93.00	9	AV9	23.7	26.5	37.672	6.85	45.0	338	207	164	150
			STD	0.4	0.5	1.150	0.11	0.3	34	12	6	5
			MAX	24.4	27.3	39.608	6.98	45.7	403	225	171	161
			@BL	103	103	104	105	106	105	99	104	103
115	94.00	9	AV9	23.4	26.9	37.875	6.71	45.4	360	189	158	151
			STD	0.5	0.5	1.946	0.16	0.5	32	12	6	3
			MAX	24.3	27.5	40.941	7.00	46.3	400	208	168	155
			@BL	108	109	108	108	114	112	109	109	114
123	95.00	8	AV8	22.7	26.8	37.127	6.58	45.9	315	192	158	155
			STD	0.8	0.8	2.057	0.18	0.6	50	9	5	5
			MAX	24.1	28.1	41.063	6.88	46.5	376	206	165	161
			@BL	117	117	116	117	119	117	117	121	120
131	96.00	8	AV8	22.2	28.1	38.289	6.57	45.9	264	206	167	143
			STD	0.4	0.6	1.442	0.12	0.4	6	13	14	9
			MAX	22.8	28.9	39.998	6.71	46.7	272	225	188	162
			@BL	124	130	124	124	127	124	125	125	125
139	97.00	8	AV8	22.0	28.9	37.751	6.61	45.8	258	179	152	140
			STD	0.5	0.8	1.425	0.15	0.5	16	12	7	10
			MAX	22.9	30.6	40.083	6.93	46.4	285	196	162	154
			@BL	137	137	137	137	135	138	132	133	133
147	98.00	8	AV8	22.1	28.8	37.697	6.62	45.7	260	167	152	139
			STD	0.4	0.6	1.195	0.14	0.4	11	7	8	12
			MAX	23.0	30.0	40.449	6.91	46.4	276	182	170	164
			@BL	144	144	144	144	146	143	142	142	142
155	99.00	8	AV8	21.7	27.9	37.020	6.46	46.3	239	166	147	129
			STD	0.6	0.9	1.706	0.18	0.6	7	7	8	12
			MAX	22.6	29.0	39.548	6.71	47.3	247	177	162	150
			@BL	150	153	155	150	149	151	150	151	151

KIEWIT GENERAL, CASTING YARD - PILE 5
OP: RMDT:--RMINER

PP24x0.401", D46-32
Test date: 21-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
163	100.00	8	AV8	22.3	28.7	38.985	6.59	45.9	256	166	148	132
			STD	0.4	0.4	0.985	0.10	0.3	16	7	7	7
			MAX	22.7	29.2	40.504	6.75	46.4	282	183	158	142
			@BL	162	163	156	162	160	162	156	156	161
170	101.00	7	AV7	22.2	28.4	38.364	6.56	46.0	258	169	152	135
			STD	0.5	0.8	1.526	0.16	0.5	6	11	9	9
			MAX	23.1	29.8	40.972	6.81	46.7	266	187	164	148
			@BL	169	169	169	169	170	164	165	165	166
177	102.00	7	AV7	22.5	28.7	38.727	6.51	46.1	244	156	131	111
			STD	0.5	0.7	1.266	0.15	0.5	4	11	11	6
			MAX	23.2	29.6	40.149	6.70	46.9	251	174	148	123
			@BL	177	177	177	177	173	174	174	174	174
185	103.00	8	AV8	22.3	28.1	37.359	6.37	46.6	242	147	125	108
			STD	0.3	0.4	1.058	0.09	0.3	9	10	14	12
			MAX	23.0	29.0	39.071	6.52	47.1	250	161	142	123
			@BL	179	179	185	179	181	181	183	183	183
192	104.00	7	AV7	22.8	29.0	38.390	6.54	46.0	254	149	129	116
			STD	0.2	0.5	1.261	0.07	0.2	7	5	9	7
			MAX	23.1	29.8	40.355	6.67	46.4	265	158	140	128
			@BL	188	188	188	188	192	192	191	192	192
199	105.00	7	AV7	22.7	28.5	37.939	6.54	46.0	262	183	149	130
			STD	0.4	0.6	1.462	0.11	0.4	10	16	9	10
			MAX	23.4	29.5	40.496	6.72	46.6	279	205	165	146
			@BL	196	196	196	196	199	198	198	198	199
206	106.00	7	AV7	23.6	29.3	38.137	6.80	45.2	347	268	227	199
			STD	0.9	1.5	1.615	0.34	1.1	68	64	60	47
			MAX	25.4	32.2	40.820	7.45	46.4	448	354	310	266
			@BL	206	206	206	206	200	206	206	206	206
214	107.00	8	AV8	24.9	31.1	39.497	7.28	43.7	435	347	306	266
			STD	0.5	0.8	0.998	0.13	0.4	13	11	14	16
			MAX	25.9	32.8	41.393	7.56	44.3	456	368	332	297
			@BL	208	208	208	208	213	208	214	214	214
221	108.00	7	AV7	24.7	30.8	39.634	7.26	43.8	403	359	322	285
			STD	0.5	0.7	1.056	0.14	0.4	2	7	9	10
			MAX	25.1	31.5	40.683	7.39	44.6	406	371	337	302
			@BL	221	221	220	220	215	216	218	218	218
228	109.00	7	AV7	25.1	31.5	41.391	7.35	43.5	392	355	313	271
			STD	0.2	0.3	0.680	0.06	0.2	4	14	16	18
			MAX	25.6	32.0	42.348	7.46	43.7	399	370	328	290
			@BL	227	224	227	227	222	223	226	227	228
236	110.00	8	AV8	24.6	30.5	39.780	7.20	44.0	375	336	290	244
			STD	0.8	1.3	1.809	0.27	0.8	10	17	18	18
			MAX	26.0	32.6	42.769	7.64	45.3	391	359	311	267
			@BL	231	231	231	231	233	231	231	229	229
248	111.00	12	AV12	24.6	30.6	40.448	7.25	43.8	385	348	304	259
			STD	0.4	0.7	0.982	0.14	0.4	8	10	11	13
			MAX	25.4	32.0	42.226	7.50	44.4	397	364	320	278
			@BL	239	239	238	239	242	245	245	245	243
259	112.00	11	AV11	24.6	30.8	39.847	7.26	43.8	408	356	317	278
			STD	0.5	0.8	1.114	0.16	0.5	13	11	13	15
			MAX	25.3	31.8	40.949	7.47	44.6	431	375	335	296
			@BL	259	259	259	259	253	259	259	259	255
270	113.00	11	AV11	25.4	31.7	41.168	7.48	43.2	437	364	321	282
			STD	0.5	0.7	1.752	0.17	0.5	7	13	15	12
			MAX	26.2	32.8	44.232	7.73	43.9	448	389	349	309
			@BL	266	266	270	266	260	266	266	266	266
281	114.00	11	AV11	25.4	31.6	41.259	7.54	43.0	453	371	325	289
			STD	0.7	1.2	1.849	0.25	0.7	12	18	21	17
			MAX	26.8	33.8	44.351	8.02	44.1	479	406	366	326
			@BL	281	281	281	281	278	281	281	281	281

KIEWIT GENERAL, CASTING YARD - PILE 5
OP: RMDT:--RMINER

PP24x0.401", D46-32
Test date: 21-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
293	115.00	12	AV12	25.5	32.2	41.777	7.65	42.7	462	390	345	311
			STD	0.5	0.8	1.583	0.18	0.5	6	12	14	13
			MAX	26.3	33.5	44.124	7.94	43.5	475	413	366	339
			@BL	288	288	288	285	282	293	293	293	293
306	116.00	13	AV13	25.8	32.6	41.173	7.71	42.5	495	478	429	392
			STD	0.5	0.7	1.132	0.15	0.4	8	17	14	15
			MAX	26.3	33.6	42.875	7.88	43.2	506	507	451	414
			@BL	296	296	299	299	302	303	304	304	306
321	117.00	15	AV15	26.2	33.2	42.612	7.89	42.1	499	488	435	391
			STD	0.3	0.5	0.986	0.12	0.3	14	41	32	22
			MAX	26.9	34.5	45.105	8.19	42.5	514	532	475	418
			@BL	312	312	312	312	317	309	310	310	310
339	118.00	18	AV18	25.5	32.3	41.325	7.67	42.6	456	405	370	347
			STD	0.5	0.7	1.234	0.16	0.4	8	11	11	11
			MAX	26.3	33.4	43.217	7.94	43.4	468	427	390	366
			@BL	324	337	337	324	325	324	338	328	334
356	119.00	17	AV17	26.2	33.0	43.027	7.85	42.2	468	428	393	367
			STD	0.7	1.0	1.800	0.21	0.6	9	13	15	15
			MAX	27.2	34.5	45.629	8.13	43.3	488	451	425	399
			@BL	347	347	354	347	341	347	356	356	356
372	120.00	16	AV16	25.9	32.8	41.886	7.75	42.4	458	429	398	368
			STD	0.6	0.8	1.798	0.19	0.5	12	13	16	17
			MAX	26.7	33.7	44.772	7.99	43.7	477	453	428	404
			@BL	362	359	361	359	370	358	358	358	358
388	121.00	16	AV16	26.0	32.9	41.955	7.75	42.4	465	439	407	378
			STD	0.5	0.8	1.406	0.16	0.4	8	12	14	16
			MAX	26.8	33.9	44.001	7.99	43.4	480	459	433	406
			@BL	381	381	385	381	379	382	382	382	382
403	122.00	15	AV15	26.1	33.0	42.247	7.77	42.4	454	435	402	371
			STD	0.6	0.9	1.558	0.19	0.5	6	13	15	17
			MAX	27.1	34.7	44.957	8.09	43.0	469	452	421	393
			@BL	403	403	390	397	394	397	389	396	396
420	123.00	17	AV17	26.1	33.0	42.229	7.79	42.3	457	431	395	362
			STD	0.6	0.9	1.562	0.20	0.5	6	11	14	18
			MAX	27.9	35.5	45.850	8.32	43.0	472	452	418	392
			@BL	418	418	418	418	412	418	406	406	406
437	124.00	17	AV17	26.3	33.0	41.547	7.80	42.3	485	464	415	375
			STD	0.6	0.8	1.429	0.18	0.5	21	36	30	22
			MAX	27.5	34.6	44.427	8.16	43.2	517	526	476	426
			@BL	433	433	433	433	427	437	431	431	431
453	125.00	16	AV16	27.3	33.9	43.488	8.02	41.7	514	500	446	400
			STD	0.5	0.7	1.555	0.15	0.4	5	25	19	12
			MAX	28.2	35.2	45.756	8.25	42.4	521	535	475	422
			@BL	450	447	447	447	452	438	445	445	449
472	126.00	19	AV19	27.1	33.7	42.786	8.00	41.8	508	498	449	414
			STD	0.8	1.0	1.789	0.21	0.5	17	34	29	26
			MAX	28.8	35.6	46.296	8.46	42.9	551	545	494	453
			@BL	471	471	454	471	468	471	455	455	469
490	127.00	18	AV18	27.2	33.7	42.616	7.98	41.8	521	514	475	446
			STD	0.4	0.6	1.055	0.12	0.3	15	40	29	26
			MAX	28.0	35.2	44.454	8.21	42.5	555	573	523	493
			@BL	476	476	475	476	482	476	474	474	478
510	128.00	20	AV20	27.5	33.8	43.779	8.03	41.7	509	493	470	449
			STD	0.4	0.4	1.014	0.09	0.2	14	23	25	27
			MAX	28.4	34.8	45.728	8.20	42.0	534	528	509	492
			@BL	500	499	499	499	497	506	506	506	506
527	129.00	17	AV17	27.7	34.0	43.690	7.97	41.8	492	511	486	467
			STD	0.6	0.8	1.577	0.14	0.3	21	12	9	10
			MAX	29.4	35.9	46.992	8.37	42.4	525	545	502	484
			@BL	514	514	514	514	512	525	512	526	526

KIEWIT GENERAL, CASTING YARD - PILE 5
OP: RMDT:--RMINER

PP24x0.401", D46-32
Test date: 21-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
544	130.00	17	AV17	27.9	34.4	43.959	7.95	41.9	469	503	481	460
			STD	0.6	0.8	1.353	0.15	0.4	25	14	16	18
			MAX	29.3	36.2	46.652	8.23	42.6	515	536	519	501
			@BL	535	535	530	530	532	534	534	534	534
562	131.00	18	AV18	27.8	34.6	43.286	7.89	42.0	443	487	459	435
			STD	0.6	0.7	1.624	0.16	0.4	8	10	9	11
			MAX	28.6	35.6	45.386	8.12	43.0	460	504	474	451
			@BL	545	551	553	551	558	558	562	545	560
579	132.00	17	AV17	28.2	35.4	44.190	7.98	41.8	449	502	462	438
			STD	0.6	0.9	1.721	0.16	0.4	13	13	9	10
			MAX	29.6	37.6	47.598	8.34	42.4	472	532	476	452
			@BL	565	565	575	565	573	566	563	568	564
596	133.00	17	AV17	28.4	35.8	43.894	7.96	41.9	462	529	475	437
			STD	0.9	1.1	2.459	0.23	0.6	14	17	18	18
			MAX	30.1	38.0	47.970	8.43	43.2	490	572	529	486
			@BL	593	593	593	593	583	590	583	583	583
615	134.00	19	AV19	28.6	36.0	44.427	8.04	41.7	467	537	481	429
			STD	0.8	1.0	1.641	0.20	0.5	9	19	20	19
			MAX	30.2	38.1	47.984	8.37	42.4	487	568	516	465
			@BL	604	604	600	604	601	606	610	610	610
634	135.00	19	AV19	29.0	36.3	44.927	8.12	41.5	477	583	527	472
			STD	0.5	0.7	1.286	0.14	0.4	9	10	13	15
			MAX	29.9	37.6	46.986	8.37	42.1	493	603	549	496
			@BL	616	616	616	616	631	632	624	624	624
654	136.00	20	AV20	28.8	36.0	44.337	8.07	41.6	493	600	549	499
			STD	0.8	1.0	1.897	0.21	0.5	14	29	34	39
			MAX	30.3	38.1	48.667	8.47	42.8	515	644	602	559
			@BL	638	638	638	638	635	649	649	649	649
676	137.00	22	AV22	29.0	36.3	43.975	8.10	41.5	508	663	620	577
			STD	0.7	0.9	1.543	0.18	0.4	14	25	28	31
			MAX	30.7	38.6	47.800	8.53	42.5	528	700	664	629
			@BL	675	675	675	675	658	673	657	657	657
698	138.00	22	AV22	29.1	36.7	44.133	8.11	41.5	515	658	616	575
			STD	0.5	0.8	1.430	0.16	0.4	21	40	46	52
			MAX	30.3	38.6	46.736	8.52	42.1	559	714	674	639
			@BL	680	680	680	680	685	694	689	689	692
723	139.00	25	AV25	29.0	36.7	43.716	8.10	41.5	536	681	646	610
			STD	0.6	0.8	1.426	0.17	0.4	10	21	21	23
			MAX	30.2	38.0	45.866	8.44	42.5	561	730	691	658
			@BL	703	703	700	703	715	700	703	703	707
749	140.00	26	AV26	28.1	36.5	42.417	8.20	41.3	583	638	603	570
			STD	1.2	1.2	2.477	0.20	0.5	22	34	35	35
			MAX	31.2	39.9	48.569	8.71	42.1	632	699	667	638
			@BL	724	724	724	724	726	734	734	732	732

Time Summary

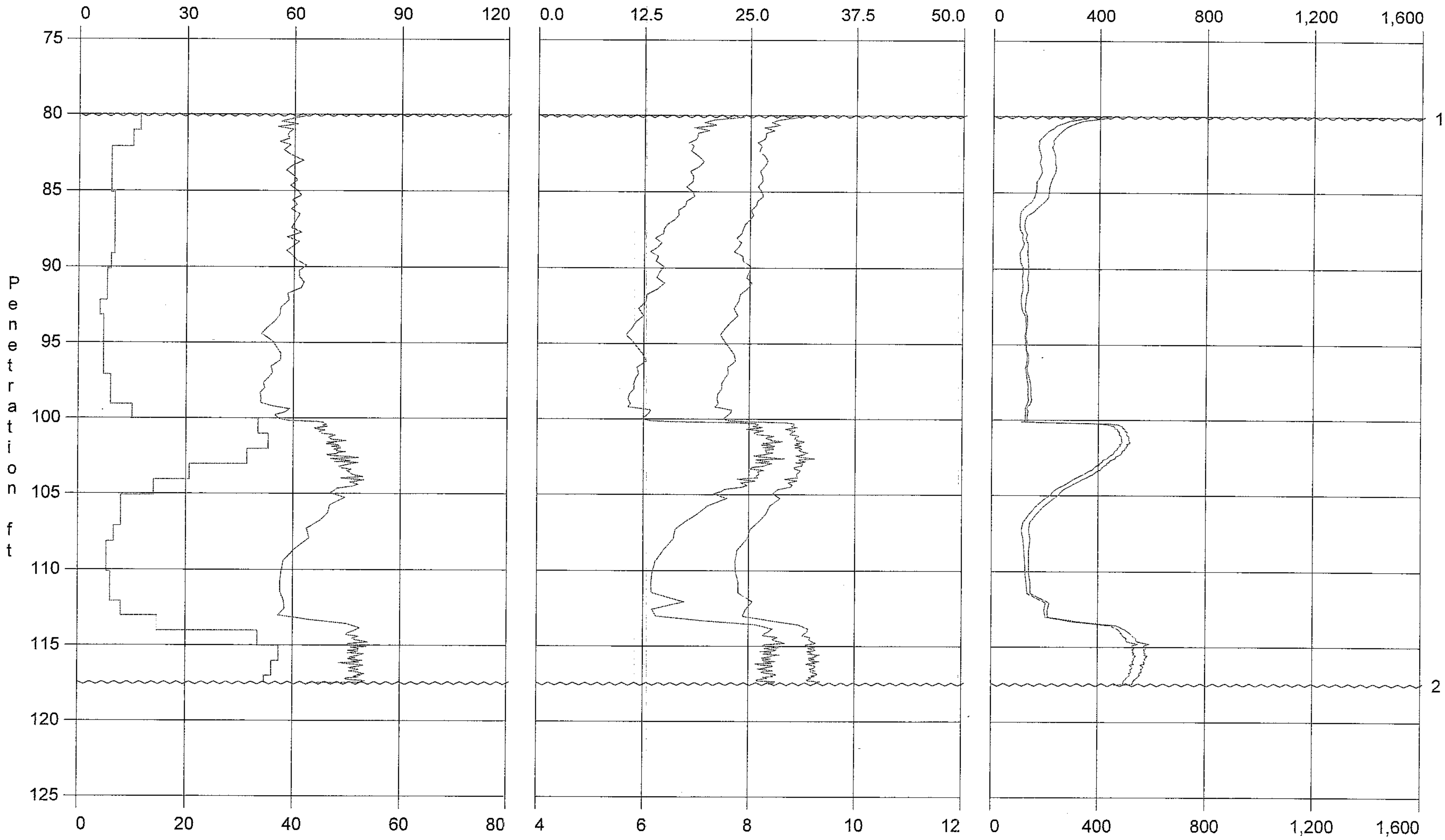
Drive 17 minutes 31 seconds 4:04:48 PM - 4:22:19 PM (4/21/2010) BN 1 - 749

PDIPLOT Ver. 2009.1 - Printed: 11-May-2010

Robert Miner Dynamic Testing, Inc. - Case Method Results

Test date: 21-Apr-2010

KIEWIT GENERAL, CASTING YARD - PILE 6 PP18x0.375", D46-32



1 - Start of test on 4/21/2010 at 1:27 PM

2 - End of test on 4/21/2010 at 1:46 PM

KIEWIT GENERAL, CASTING YARD - PILE 6
OP: RMDT:--RMINER

PP18x0.375", D46-32
Test date: 21-Apr-2010

AR: 20.76 in^2
LE: 119.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.40

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
EMX: Max Transferred Energy
STK: O.E. Diesel Hammer Stroke
BPM: Blows per Minute

RP1: Case-Goble Capacity (JC=0.1)
RX4: Max Case Method Capacity (JC=0.4)
RX6: Max Case Method Capacity (JC=0.6)
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
17	81.00	17	AV17	28.3	30.7	39.296	7.31	43.7	545	396	318	265
			STD	1.8	2.0	3.444	0.48	1.2	64	69	60	51
			MAX	34.1	36.0	49.904	8.99	45.3	736	595	491	393
			@BL	1	1	1	1	13	1	1	1	1
32	82.00	15	AV15	26.5	28.6	38.545	6.96	44.7	447	299	234	181
			STD	0.7	0.9	1.350	0.16	0.5	20	11	11	12
			MAX	27.9	30.5	41.890	7.27	45.2	490	328	257	202
			@BL	26	26	26	26	28	18	18	18	18
41	83.00	9	AV9	26.6	28.3	39.777	7.01	44.5	436	297	231	176
			STD	0.6	0.8	2.068	0.15	0.5	14	5	6	5
			MAX	27.5	29.3	43.830	7.25	45.6	455	303	237	182
			@BL	40	40	40	40	35	40	40	36	38
50	84.00	9	AV9	26.3	28.6	39.147	6.93	44.7	434	296	233	182
			STD	0.5	0.5	1.057	0.13	0.4	9	3	4	4
			MAX	27.3	29.5	41.386	7.19	45.2	452	302	238	187
			@BL	43	43	43	43	45	43	43	44	45
59	85.00	9	AV9	26.2	28.9	39.949	6.87	44.9	405	279	214	164
			STD	0.7	0.8	1.117	0.14	0.4	12	5	4	3
			MAX	27.4	30.1	41.762	7.15	45.5	427	288	222	170
			@BL	53	53	53	53	59	53	51	53	52
69	86.00	10	AV10	26.1	29.0	40.570	6.85	45.0	403	261	193	142
			STD	0.5	0.6	1.300	0.10	0.3	10	16	16	16
			MAX	26.8	29.9	42.174	7.02	45.5	415	281	212	163
			@BL	61	60	61	61	65	61	61	61	60
79	87.00	10	AV10	25.0	27.4	40.195	6.60	45.8	356	199	133	102
			STD	0.9	1.3	1.821	0.20	0.7	18	20	16	3
			MAX	26.2	28.9	42.812	6.88	46.9	388	231	160	107
			@BL	74	70	76	70	71	70	70	70	71
89	88.00	10	AV10	24.1	24.9	40.111	6.37	46.6	311	159	123	107
			STD	0.6	0.9	1.306	0.14	0.5	13	7	5	7
			MAX	25.2	26.7	42.881	6.64	47.3	339	173	133	117
			@BL	80	80	86	80	89	80	80	86	86
99	89.00	10	AV10	23.5	24.4	39.519	6.22	47.1	289	161	130	109
			STD	0.6	0.7	1.377	0.14	0.5	7	5	4	7
			MAX	24.7	25.7	42.124	6.48	47.7	302	169	133	121
			@BL	92	92	92	92	99	92	98	96	92
108	90.00	9	AV9	24.5	25.5	40.798	6.29	46.9	301	168	133	107
			STD	0.5	0.6	1.380	0.09	0.3	6	3	3	5
			MAX	25.3	26.5	42.786	6.42	47.4	309	172	139	117
			@BL	106	106	106	106	105	102	104	108	108
116	91.00	8	AV8	24.9	26.1	41.151	6.31	46.8	291	168	133	112
			STD	0.5	0.6	1.069	0.12	0.4	7	3	3	2
			MAX	25.5	26.7	43.219	6.46	47.5	299	171	136	114
			@BL	114	114	114	114	112	116	114	111	116
124	92.00	8	AV8	24.3	25.3	40.197	6.17	47.3	281	161	131	110
			STD	0.7	0.8	1.747	0.15	0.6	10	4	5	4
			MAX	25.2	26.3	42.140	6.36	48.2	295	168	140	116
			@BL	120	120	120	120	123	117	117	120	120
130	93.00	6	AV6	23.3	24.4	38.101	5.95	48.1	271	153	124	111
			STD	0.7	0.8	1.178	0.14	0.5	8	7	6	8
			MAX	24.3	25.4	39.473	6.13	48.9	285	161	132	122
			@BL	128	128	128	126	127	128	126	128	130

KIEWIT GENERAL, CASTING YARD - PILE 6
OP: RMDT:--RMINER

PP18x0.375", D46-32
Test date: 21-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
137	94.00	7	AV7	22.9	24.0	36.322	5.88	48.4	258	145	130	126
			STD	0.7	0.8	0.857	0.11	0.4	12	4	3	3
			MAX	24.1	25.4	37.756	6.11	48.9	280	149	133	130
			@BL	131	131	131	131	137	131	131	134	134
144	95.00	7	AV7	21.8	22.6	34.958	5.73	49.0	239	140	128	125
			STD	0.5	0.6	1.081	0.10	0.4	5	5	3	3
			MAX	22.4	23.2	36.282	5.84	49.8	244	149	131	129
			@BL	143	140	142	143	141	143	143	138	138
151	96.00	7	AV7	22.9	23.5	37.058	5.93	48.2	250	145	132	129
			STD	0.5	0.8	0.897	0.11	0.4	9	4	5	4
			MAX	23.9	25.0	38.771	6.15	48.7	264	150	140	137
			@BL	150	150	150	150	146	150	150	149	149
158	97.00	7	AV7	22.8	23.4	36.522	5.94	48.2	249	148	131	127
			STD	0.7	0.8	1.308	0.12	0.5	10	5	4	4
			MAX	23.7	24.7	38.483	6.14	48.7	264	155	136	131
			@BL	153	152	152	153	155	152	153	153	152
167	98.00	9	AV9	22.0	22.3	35.110	5.85	48.5	230	161	144	134
			STD	0.5	0.6	0.756	0.09	0.3	9	6	6	5
			MAX	22.7	23.4	36.457	5.94	49.3	247	167	152	142
			@BL	159	159	159	159	165	159	167	166	166
176	99.00	9	AV9	21.2	21.4	33.827	5.73	49.0	214	161	145	134
			STD	0.5	0.5	0.634	0.08	0.3	6	7	8	7
			MAX	21.8	22.1	34.890	5.81	49.7	223	169	155	144
			@BL	175	175	172	173	174	175	172	172	172
191	100.00	15	AV15	22.1	22.4	37.097	5.96	48.1	261	150	133	125
			STD	1.0	1.0	1.950	0.22	0.8	27	7	4	3
			MAX	23.6	23.9	39.865	6.28	49.6	297	162	139	131
			@BL	183	183	182	183	178	186	182	186	186
241	101.00	50	AV50	28.6	30.6	43.817	7.73	42.6	509	467	425	397
			STD	2.8	3.8	3.133	0.77	2.3	110	144	136	131
			MAX	31.9	35.2	49.043	8.58	48.1	585	560	514	485
			@BL	217	229	217	217	195	229	238	229	238
294	102.00	53	AV53	30.8	33.9	47.614	8.39	40.8	581	555	509	481
			STD	0.6	1.1	1.440	0.16	0.4	9	8	7	7
			MAX	31.8	35.9	50.476	8.67	41.6	601	571	522	492
			@BL	271	292	265	266	286	281	265	265	263
341	103.00	47	AV47	31.3	34.2	48.926	8.31	41.0	561	510	461	432
			STD	0.8	1.0	2.472	0.22	0.5	10	20	22	22
			MAX	33.4	37.1	57.047	8.86	42.0	580	543	492	462
			@BL	324	324	339	339	336	324	305	305	299
372	104.00	31	AV31	30.6	34.6	50.248	8.13	41.5	526	431	383	350
			STD	0.7	0.9	2.032	0.19	0.5	17	26	26	29
			MAX	31.9	36.1	55.903	8.50	42.9	555	470	421	392
			@BL	352	369	369	352	372	343	343	342	342
393	105.00	21	AV21	29.2	33.1	49.932	7.74	42.5	471	342	284	254
			STD	1.1	1.2	2.530	0.29	0.8	22	25	25	25
			MAX	31.0	35.2	54.316	8.19	43.9	506	384	330	297
			@BL	375	375	375	375	391	374	373	373	373
405	106.00	12	AV6	28.0	31.2	48.210	7.40	43.4	425	266	212	181
			STD	0.7	1.3	1.805	0.21	0.6	19	27	21	20
			MAX	29.0	33.3	51.206	7.70	44.2	457	305	246	215
			@BL	394	394	396	394	402	394	394	394	394
416	107.00	12	AV6	26.6	28.6	45.738	6.92	44.8	374	200	155	129
			STD	0.6	0.7	1.774	0.17	0.5	12	9	13	12
			MAX	27.5	29.8	48.340	7.16	45.6	391	217	175	147
			@BL	406	406	406	406	414	406	406	406	406
426	108.00	10	AV5	25.1	26.7	42.761	6.60	45.8	342	176	142	114
			STD	0.6	0.9	1.557	0.15	0.5	13	5	4	3
			MAX	25.7	27.8	44.672	6.75	46.5	356	181	147	118
			@BL	420	420	424	424	422	418	424	424	424

KIEWIT GENERAL, CASTING YARD - PILE 6
OP: RMDT:--RMINER

PP18x0.375", D46-32
Test date: 21-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
435	109.00	8	AV4	23.8	24.7	40.882	6.43	46.4	304	174	141	122
			STD	0.4	0.6	1.168	0.09	0.3	9	6	3	1
			MAX	24.4	25.4	42.732	6.55	46.8	317	180	145	122
			@BL	428	428	428	428	432	428	428	428	434
443	110.00	8	AV4	23.3	24.3	37.845	6.22	47.1	289	166	139	126
			STD	0.4	0.4	0.671	0.06	0.2	3	4	2	2
			MAX	24.1	25.0	38.503	6.32	47.3	293	171	141	127
			@BL	440	440	440	440	442	440	436	436	438
452	111.00	9	AV5	23.7	24.0	37.912	6.17	47.3	274	165	143	128
			STD	0.4	0.4	0.798	0.06	0.2	5	5	5	3
			MAX	24.1	24.4	39.357	6.28	47.6	281	169	148	133
			@BL	446	446	446	446	444	446	448	450	452
461	112.00	9	AV4	24.2	25.1	37.803	6.60	46.0	289	183	165	153
			STD	0.9	1.1	0.984	0.77	2.4	14	32	34	33
			MAX	25.6	27.0	38.707	7.92	47.7	311	238	224	211
			@BL	460	460	454	460	456	460	460	460	460
473	113.00	12	AV6	24.8	26.6	38.076	6.19	47.2	301	226	213	201
			STD	0.8	0.9	1.627	0.10	0.4	5	4	2	4
			MAX	25.9	27.5	40.805	6.35	48.0	309	232	217	207
			@BL	466	466	466	466	468	466	462	462	472
495	114.00	22	AV11	29.1	31.7	46.486	7.60	43.0	443	429	392	370
			STD	2.9	3.3	5.904	0.85	2.4	84	126	116	110
			MAX	32.0	35.3	52.625	8.45	47.3	558	564	511	485
			@BL	492	492	494	492	476	494	494	494	494
545	115.00	50	AV29	32.0	35.5	51.225	8.41	40.8	592	598	552	514
			STD	0.6	1.0	1.828	0.19	0.4	41	23	24	20
			MAX	33.5	37.6	54.968	8.79	41.7	695	641	596	553
			@BL	535	535	535	535	542	534	535	535	534
601	116.00	56	AV56	32.5	35.1	51.737	8.39	40.8	618	620	576	534
			STD	0.7	1.0	1.676	0.18	0.4	15	7	6	5
			MAX	33.9	37.0	55.313	8.74	41.9	655	639	594	549
			@BL	579	559	578	579	573	579	578	578	578
655	117.00	54	AV54	32.6	34.9	51.687	8.35	40.9	633	606	562	523
			STD	0.7	1.1	1.655	0.18	0.4	15	8	8	7
			MAX	34.3	37.0	54.628	8.69	42.4	665	627	581	539
			@BL	624	654	620	607	609	607	607	607	607
681	117.50	52	AV26	32.6	35.1	51.344	8.33	41.0	614	580	534	498
			STD	0.7	0.7	2.242	0.15	0.4	11	10	9	9
			MAX	33.9	36.5	54.425	8.57	41.8	631	591	546	508
			@BL	678	656	673	673	668	672	656	656	660

Time Summary

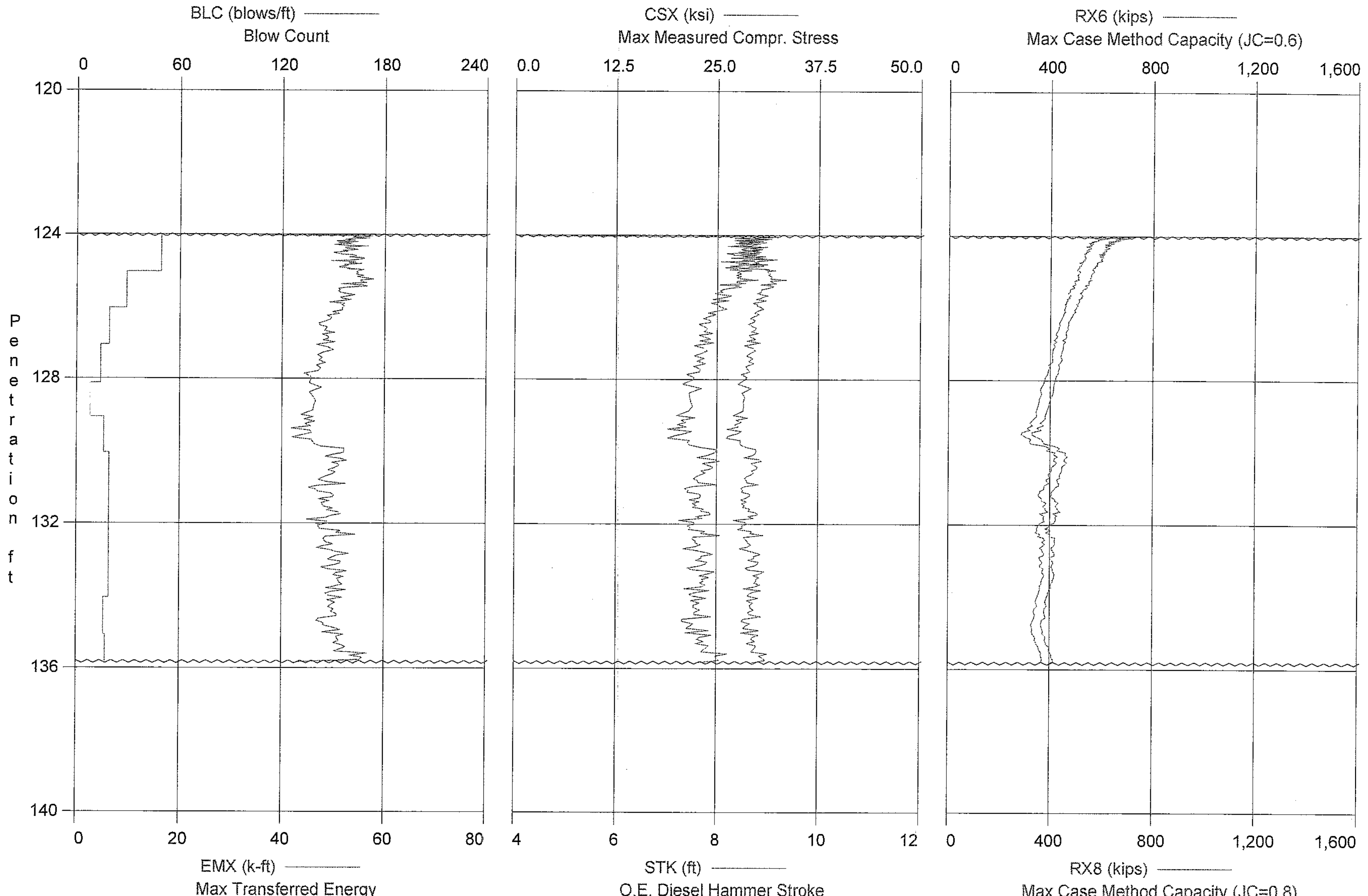
Drive 19 minutes 36 seconds 1:27:04 PM - 1:46:40 PM (4/21/2010) BN 1 - 681

PDIPLOT Ver. 2009.1 - Printed: 11-May-2010

Robert Miner Dynamic Testing, Inc. - Case Method Results

Test date: 21-Apr-2010

KIEWIT GENERAL, CASTING YARD - PILE 7 PP18x0.375", D46-32



1 - Start of test on 4/21/2010 at 3:00 PM

2 - End of test on 4/21/2010 at 3:06 PM

KIEWIT GENERAL, CASTING YARD - PILE 7
OP: RMDT:--RMINER

PP18x0.375", D46-32
Test date: 21-Apr-2010

AR: 20.76 in^2
LE: 165.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.40

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
EMX: Max Transferred Energy
STK: O.E. Diesel Hammer Stroke
BPM: Blows per Minute

RP1: Case-Goble Capacity (JC=0.1)
RX4: Max Case Method Capacity (JC=0.4)
RX6: Max Case Method Capacity (JC=0.6)
RX8: Max Case Method Capacity (JC=0.8)

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
49	125.00	49	AV49	29.7	45.0	52.875	8.54	40.5	765	685	605	539
			STD	1.7	2.7	3.272	0.24	0.5	52	36	34	28
			MAX	32.7	47.8	57.574	9.25	41.6	915	791	710	629
			@BL	2	32	2	2	36	2	2	2	2
78	126.00	29	AV29	30.9	37.3	53.367	8.23	41.2	657	594	531	482
			STD	0.9	2.5	2.099	0.22	0.5	38	36	20	18
			MAX	33.5	41.2	57.620	8.80	42.1	721	653	561	511
			@BL	56	54	56	56	75	50	50	50	55
97	127.00	19	AV19	29.6	33.6	49.190	7.83	42.2	563	520	471	434
			STD	0.5	0.8	1.300	0.13	0.3	15	13	13	13
			MAX	30.7	35.9	51.741	8.18	42.7	596	546	498	457
			@BL	79	97	80	79	87	80	80	80	80
111	128.00	14	AV14	28.9	34.4	47.176	7.62	42.7	523	488	436	399
			STD	0.5	0.6	1.388	0.11	0.3	17	9	10	12
			MAX	29.7	35.7	49.960	7.80	43.4	541	500	450	418
			@BL	100	105	100	100	109	100	99	98	98
119	129.00	8	AV8	28.1	33.8	45.858	7.44	43.2	507	453	399	357
			STD	0.6	0.7	1.047	0.13	0.4	7	12	12	9
			MAX	29.2	35.4	47.626	7.68	43.9	518	469	413	371
			@BL	113	113	113	113	119	112	113	112	112
135	130.00	16	AV16	27.8	33.0	45.964	7.43	43.3	475	430	379	336
			STD	1.0	2.0	2.762	0.27	0.7	21	34	37	38
			MAX	29.8	36.8	52.116	7.98	44.5	522	511	461	419
			@BL	135	134	135	135	125	135	135	135	135
154	131.00	19	AV19	29.2	34.1	49.648	7.74	42.4	525	497	443	401
			STD	0.7	1.4	1.920	0.18	0.5	13	15	14	16
			MAX	30.5	36.6	52.597	8.05	43.4	545	521	468	429
			@BL	140	136	140	140	154	141	140	137	137
173	132.00	19	AV19	28.7	32.3	48.739	7.60	42.8	495	472	418	371
			STD	0.7	0.7	1.548	0.15	0.4	8	10	11	11
			MAX	30.1	33.4	51.600	7.87	43.8	508	490	439	391
			@BL	168	168	168	167	171	167	166	166	166
192	133.00	19	AV19	29.0	31.9	49.454	7.63	42.7	484	461	407	365
			STD	0.8	1.0	1.989	0.17	0.5	6	11	11	11
			MAX	30.7	34.0	54.416	8.07	43.5	494	476	420	380
			@BL	179	179	179	179	186	181	180	180	183
211	134.00	19	AV19	29.7	32.2	50.496	7.68	42.6	487	453	406	365
			STD	0.7	0.9	1.408	0.16	0.4	8	11	9	7
			MAX	30.9	33.7	52.809	7.97	43.5	502	466	419	377
			@BL	198	198	198	198	207	200	200	199	199
227	135.00	16	AV16	29.1	30.6	49.481	7.57	42.9	461	419	377	338
			STD	0.7	0.9	1.435	0.17	0.5	7	7	7	7
			MAX	30.5	32.3	51.896	7.89	43.6	476	429	388	350
			@BL	220	220	212	220	222	212	212	215	215
241	135.83	17	AV14	29.9	31.0	51.813	7.82	42.2	478	442	396	356
			STD	0.7	1.1	3.112	0.18	0.5	10	13	11	10
			MAX	31.3	33.2	56.691	8.21	42.8	491	461	415	370
			@BL	237	237	237	237	232	237	240	240	239

Time Summary

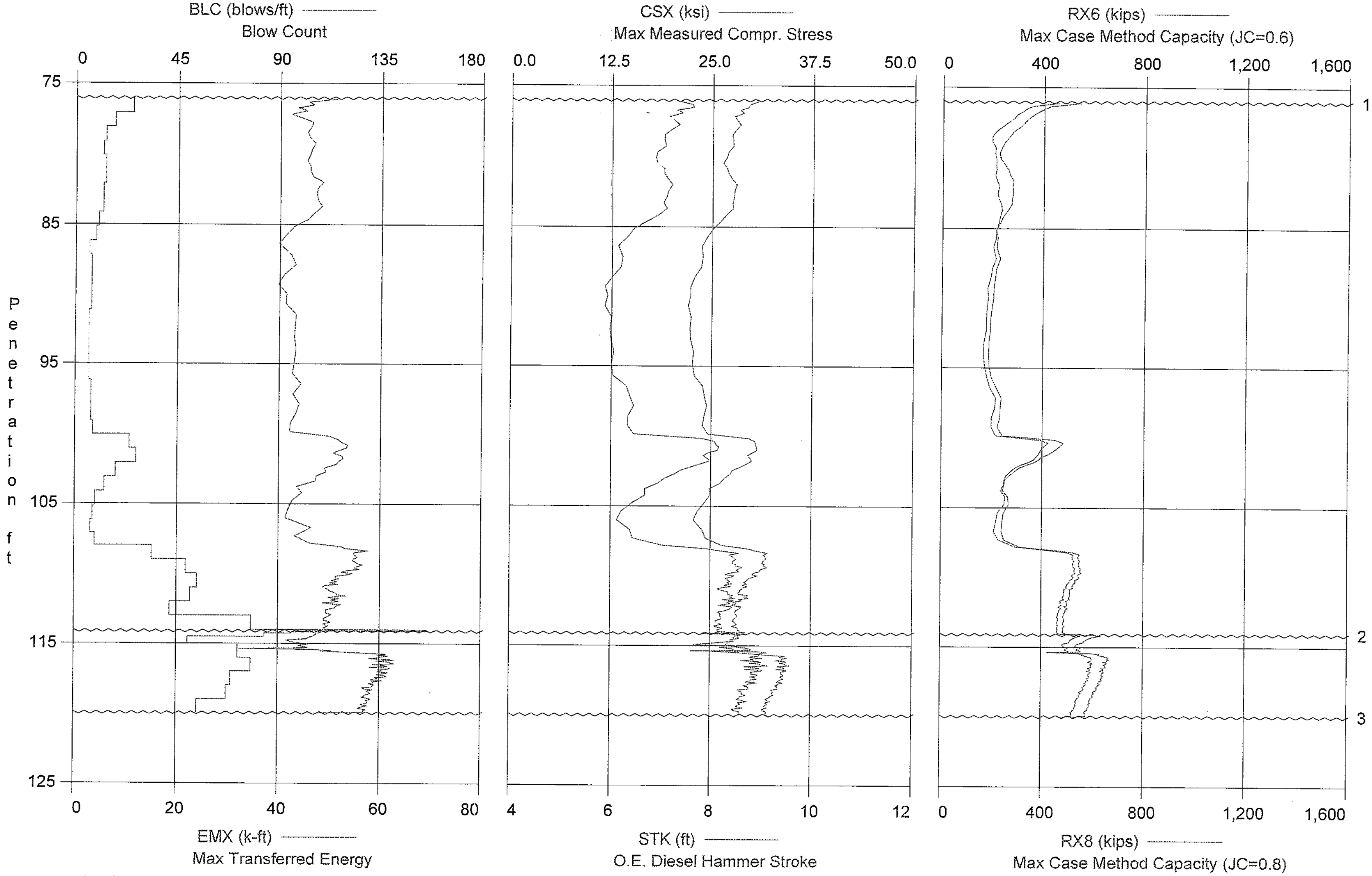
Drive 5 minutes 43 seconds 3:00:20 PM - 3:06:03 PM (4/21/2010) BN 1 - 241

PDIPLOT Ver. 2009.1 - Printed: 11-May-2010

Robert Miner Dynamic Testing, Inc. - Case Method Results

Test date: 21-Apr-2010

KIEWIT GENERAL, CASTING YARD - PILE 8 PP20x0.375", D46-32



1 - Start of test on 4/21/2010 at 2:24 PM
2 - Restart after 20 hours.
3 - End of test on 4/22/2010 at 11:11 AM

KIEWIT GENERAL, CASTING YARD - PILE 8
OP: RMDT:--RMINER

PP20x0.375", D46-32
Test date: 21-Apr-2010

AR: 23.12 in^2
LE: 140.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.40

CSX: Max Measured Compr. Stress
CSI: Max F1 or F2 Compr. Stress
EMX: Max Transferred Energy
STK: O.E. Diesel Hammer Stroke
BPM: Blows per Minute

RP1: Case-Goble Capacity (JC=0.1)
RX4: Max Case Method Capacity (JC=0.4)
RX6: Max Case Method Capacity (JC=0.6)
RX8: Max Case Method Capacity (JC=0.8)

BL#	depth	BLC	TYPE	CSX	CSI	EMX	STK	BPM	RP1	RX4	RX6	RX8
end	ft	bl/ft		ksi	ksi	k-ft	ft	**	kips	kips	kips	kips
25	77.00	25	AV25	29.3	40.9	46.304	7.59	42.9	708	535	421	349
			STD	2.0	2.5	5.441	0.35	0.9	66	70	74	64
			MAX	36.6	49.1	67.057	8.74	44.2	968	798	684	570
			@BL	2	2	2	2	19	2	2	2	2
42	78.00	17	AV17	27.9	38.7	44.302	7.21	43.9	609	423	320	255
			STD	0.7	1.1	2.208	0.19	0.6	19	22	19	22
			MAX	28.8	40.4	48.764	7.50	44.7	638	461	346	288
			@BL	37	36	36	36	38	26	26	26	26
55	79.00	13	AV13	27.6	38.0	45.870	7.09	44.3	556	363	261	199
			STD	0.8	1.3	1.919	0.22	0.6	23	21	14	7
			MAX	29.8	41.2	50.464	7.63	45.1	608	400	284	213
			@BL	45	45	45	45	52	45	45	45	43
67	80.00	12	AV12	27.1	36.9	46.273	6.95	44.7	529	341	229	208
			STD	0.7	1.0	1.444	0.16	0.5	13	12	9	6
			MAX	28.1	38.4	49.138	7.18	45.7	554	370	248	218
			@BL	57	57	65	57	62	58	58	58	65
80	81.00	13	AV13	26.6	36.6	45.659	6.95	44.7	512	332	248	212
			STD	0.7	1.2	1.983	0.19	0.6	10	9	15	7
			MAX	27.8	38.8	49.481	7.27	45.5	531	345	278	227
			@BL	76	74	76	76	75	76	79	79	76
93	82.00	13	AV13	27.3	37.0	46.493	7.09	44.3	543	359	275	214
			STD	0.5	0.8	1.507	0.14	0.4	9	6	9	8
			MAX	28.3	38.4	49.073	7.38	44.9	559	366	292	231
			@BL	92	90	90	90	81	90	88	92	92
105	83.00	12	AV12	27.8	36.8	47.645	7.12	44.2	533	350	275	221
			STD	0.5	0.6	1.654	0.11	0.3	8	8	8	5
			MAX	28.4	37.5	49.982	7.28	44.8	543	360	290	229
			@BL	95	95	95	95	96	103	99	99	103
117	84.00	12	AV12	27.3	35.8	47.232	7.02	44.5	509	332	262	232
			STD	0.7	1.0	1.741	0.16	0.5	13	14	15	9
			MAX	28.2	37.3	49.678	7.24	45.1	532	355	289	254
			@BL	115	106	112	115	117	106	106	106	106
127	85.00	10	AV10	26.2	33.7	46.110	6.75	45.3	471	291	230	224
			STD	0.9	1.7	2.086	0.23	0.7	20	13	10	5
			MAX	28.5	37.8	51.311	7.34	46.3	516	322	244	233
			@BL	119	119	119	119	126	119	119	120	119
136	86.00	9	AV9	24.5	30.8	41.507	6.34	46.7	424	250	217	212
			STD	0.4	0.6	0.830	0.09	0.3	9	12	8	3
			MAX	25.1	31.7	42.838	6.49	47.3	442	273	233	218
			@BL	135	135	135	129	134	129	129	135	135
142	87.00	6	AV6	23.5	29.2	39.953	6.10	47.6	392	235	216	202
			STD	0.3	0.3	1.017	0.05	0.2	6	7	5	2
			MAX	24.0	29.8	41.653	6.18	47.9	404	241	223	205
			@BL	139	139	142	139	137	139	139	139	139
149	88.00	7	AV7	23.7	29.4	42.619	6.22	47.1	390	250	227	209
			STD	0.7	0.9	1.958	0.14	0.5	10	11	11	10
			MAX	24.5	30.4	45.624	6.40	47.9	405	266	241	222
			@BL	143	143	149	143	146	143	143	143	145
156	89.00	7	AV7	23.0	27.7	41.514	6.04	47.8	361	233	209	191
			STD	0.7	1.0	2.487	0.14	0.5	14	6	7	6
			MAX	24.3	29.4	46.338	6.28	48.7	384	245	220	198
			@BL	150	150	150	150	153	150	152	152	152

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PP20x0.375", D46-32
Test date: 21-Apr-2010

BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
163	90.00	7	AV7	22.3	25.9	40.423	5.88	48.4	417	280	206	181
			STD	0.5	0.8	1.844	0.10	0.4	50	39	5	6
			MAX	23.2	27.1	42.409	6.02	49.0	454	309	213	189
			@BL	158	158	162	158	160	157	157	157	163
170	91.00	7	AV7	22.1	26.2	41.260	5.88	48.4	392	265	201	181
			STD	0.6	1.0	1.502	0.13	0.5	53	34	5	6
			MAX	23.2	27.8	43.775	6.12	48.9	442	299	207	190
			@BL	166	166	165	166	169	164	164	164	164
176	92.00	6	AV6	22.3	27.0	42.982	5.99	48.0	349	227	194	177
			STD	0.6	0.9	1.160	0.11	0.4	39	28	4	4
			MAX	23.2	28.2	45.210	6.16	48.5	434	287	200	185
			@BL	173	173	173	173	172	176	176	171	171
182	93.00	6	AV6	22.5	27.1	43.842	6.01	47.9	421	275	194	175
			STD	0.6	1.0	1.565	0.12	0.5	38	32	4	3
			MAX	23.4	28.9	45.279	6.19	48.5	454	302	200	179
			@BL	179	179	182	179	177	182	182	182	178
188	94.00	6	AV6	22.3	26.4	42.687	5.98	48.0	433	279	184	165
			STD	0.8	1.1	2.205	0.17	0.7	20	17	8	6
			MAX	23.5	28.0	45.901	6.24	49.1	459	302	197	173
			@BL	186	186	188	186	187	186	186	186	183
194	95.00	6	AV6	22.5	26.4	42.494	5.98	48.0	442	289	189	169
			STD	0.8	1.3	2.118	0.19	0.7	16	12	8	6
			MAX	23.5	28.2	44.953	6.23	49.2	464	306	200	180
			@BL	193	189	193	189	191	193	193	193	193
200	96.00	6	AV6	23.0	27.0	43.025	6.08	47.6	445	287	196	179
			STD	0.4	0.8	1.058	0.10	0.4	10	11	8	6
			MAX	23.7	28.7	45.156	6.29	48.2	458	301	211	187
			@BL	195	195	195	195	196	200	200	200	200
207	97.00	7	AV7	24.1	28.6	44.129	6.35	46.7	469	317	223	197
			STD	0.6	0.9	1.284	0.14	0.5	54	36	17	10
			MAX	24.8	30.0	46.541	6.50	47.5	513	354	249	209
			@BL	204	204	204	204	201	207	207	207	207
214	98.00	7	AV7	24.2	29.6	43.265	6.40	46.5	454	304	235	216
			STD	0.6	1.1	1.824	0.15	0.5	55	49	9	7
			MAX	24.9	31.1	46.045	6.58	47.4	508	349	245	223
			@BL	212	212	214	212	210	211	208	208	211
221	99.00	7	AV7	24.2	29.2	43.286	6.36	46.6	494	337	232	201
			STD	0.6	1.1	2.341	0.16	0.6	10	8	8	7
			MAX	25.1	30.6	47.584	6.59	47.5	507	346	240	213
			@BL	220	220	220	220	218	220	217	216	217
229	100.00	8	AV8	24.0	29.2	41.667	6.34	46.7	462	304	225	199
			STD	0.8	1.4	1.616	0.18	0.6	30	30	7	7
			MAX	25.7	32.0	44.168	6.69	47.8	484	331	238	216
			@BL	229	229	229	229	227	226	227	229	229
253	101.00	24	AV24	30.2	39.7	51.830	8.00	41.8	632	520	457	399
			STD	1.2	2.0	2.892	0.34	0.9	39	38	40	33
			MAX	31.7	42.2	55.833	8.39	44.9	662	555	491	427
			@BL	250	250	250	247	230	250	235	235	235
280	102.00	27	AV27	29.9	40.6	51.889	7.91	42.0	619	452	387	360
			STD	0.7	1.0	1.880	0.19	0.5	14	32	30	25
			MAX	31.4	42.4	55.171	8.27	43.8	643	503	438	395
			@BL	257	275	261	257	263	257	254	256	256
298	103.00	18	AV18	27.7	38.0	49.427	7.36	43.5	547	360	283	271
			STD	1.0	1.4	1.898	0.25	0.7	26	25	21	17
			MAX	29.1	39.7	52.371	7.73	45.6	587	395	321	302
			@BL	283	283	283	283	297	283	281	281	281
311	104.00	13	AV13	25.6	34.5	45.701	6.86	45.0	483	303	249	244
			STD	1.1	1.6	2.566	0.23	0.7	25	20	5	7
			MAX	27.3	37.3	49.182	7.24	46.4	524	329	260	256
			@BL	299	299	299	299	310	299	299	299	299

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PP20x0.375", D46-32
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BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
320	105.00	9	AV9	24.5	32.3	43.500	6.55	46.0	427	286	268	254
			STD	0.6	0.9	1.351	0.18	0.6	19	11	7	4
			MAX	25.2	33.3	45.139	6.77	46.9	452	304	280	259
			@BL	315	313	312	315	318	312	317	317	315
328	106.00	8	AV8	23.1	30.6	41.612	6.18	47.3	379	275	246	223
			STD	0.4	0.5	0.991	0.08	0.3	10	6	6	7
			MAX	23.7	31.4	42.950	6.29	47.8	394	285	257	236
			@BL	322	325	325	325	326	322	322	321	321
335	107.00	7	AV7	23.7	31.6	45.008	6.32	46.8	375	272	242	213
			STD	0.7	1.1	2.569	0.18	0.6	7	7	8	8
			MAX	24.8	33.3	47.993	6.62	47.5	387	283	254	225
			@BL	335	335	335	335	330	335	335	335	335
344	108.00	9	AV9	25.1	33.5	44.376	6.67	45.6	402	294	272	252
			STD	1.1	1.7	2.097	0.32	1.0	32	33	32	34
			MAX	27.3	36.8	48.889	7.29	46.9	468	369	341	316
			@BL	344	344	344	344	336	344	344	344	344
378	109.00	34	AV34	31.2	44.4	54.673	8.33	41.0	636	564	527	502
			STD	1.3	2.5	2.989	0.34	0.8	45	41	43	44
			MAX	33.1	47.4	60.823	8.85	43.3	687	596	561	539
			@BL	358	358	358	358	346	371	358	371	371
427	110.00	49	AV49	31.2	45.8	54.867	8.48	40.6	663	577	549	527
			STD	0.8	1.1	2.277	0.20	0.5	14	8	9	9
			MAX	32.7	47.7	59.698	8.86	41.9	690	593	567	543
			@BL	396	415	396	396	383	415	396	396	396
481	111.00	54	AV54	29.5	45.9	51.222	8.33	41.0	634	552	528	504
			STD	0.7	1.0	2.151	0.18	0.4	15	14	13	13
			MAX	31.1	47.9	56.873	8.71	42.2	675	588	561	537
			@BL	430	451	430	450	476	430	430	430	430
532	112.00	51	AV51	28.8	46.5	51.029	8.37	40.9	610	534	508	483
			STD	0.9	1.2	2.391	0.22	0.5	16	11	10	9
			MAX	30.5	48.8	56.242	8.81	41.9	642	554	525	500
			@BL	504	531	531	531	526	504	491	483	483
574	113.00	42	AV42	28.2	45.8	50.189	8.25	41.1	595	516	492	468
			STD	0.7	1.0	1.962	0.21	0.5	13	9	8	8
			MAX	30.0	47.9	55.121	8.77	42.0	633	544	514	490
			@BL	546	546	543	546	559	546	543	543	543
613	114.00	78	AV39	28.2	45.7	49.570	8.17	41.4	579	513	488	464
			STD	0.7	1.1	1.916	0.19	0.5	11	8	7	5
			MAX	29.5	47.8	52.977	8.53	42.2	605	535	504	477
			@BL	610	576	575	576	579	576	600	575	575
689	115.00	25	AV66	28.0	39.0	46.683	8.51	40.5	791	659	571	512
			STD	1.4	3.6	4.166	0.25	0.6	97	67	45	30
			MAX	29.8	47.9	52.850	9.18	42.4	892	747	649	562
			@BL	621	621	621	635	679	635	635	635	634
786	116.00	72	AV78	29.9	35.0	50.865	8.78	40.0	745	706	599	542
			STD	3.5	3.1	8.467	0.46	1.3	136	68	60	53
			MAX	35.1	39.3	63.728	9.46	50.3	888	800	677	605
			@BL	774	774	774	738	725	767	774	774	768
864	117.00	78	AV78	34.0	37.7	60.742	8.89	39.7	840	763	647	592
			STD	0.7	0.8	2.197	0.22	0.5	21	14	10	8
			MAX	35.7	39.5	65.741	9.46	40.9	908	789	665	605
			@BL	850	850	850	850	792	810	795	795	801
933	118.00	69	AV69	33.7	38.1	59.761	8.83	39.8	813	735	631	578
			STD	0.6	0.7	1.902	0.17	0.4	17	14	10	10
			MAX	35.1	39.7	64.398	9.23	40.6	857	767	653	598
			@BL	890	913	872	872	926	866	872	872	872
1000	119.00	67	AV67	32.7	36.3	57.651	8.64	40.3	793	697	603	550
			STD	0.7	0.8	1.726	0.17	0.4	18	13	11	10
			MAX	34.2	38.1	61.115	8.97	41.4	835	723	626	573
			@BL	942	947	967	937	946	955	947	936	936

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BL# end	depth ft	BLC bl/ft	TYPE	CSX ksi	CSI ksi	EMX k-ft	STK ft	BPM **	RP1 kips	RX4 kips	RX6 kips	RX8 kips
1054	120.00	54	AV54	31.9	35.6	56.916	8.57	40.4	792	673	581	527
			STD	0.4	0.5	1.738	0.12	0.3	18	9	11	11
			MAX	33.0	37.0	60.385	8.89	41.0	842	697	599	545
			@BL	1003	1003	1011	1003	1033	1003	1003	1011	1011

BL#	depth (ft)	Comments
725	115.37	Stop and Fresh-head pile.

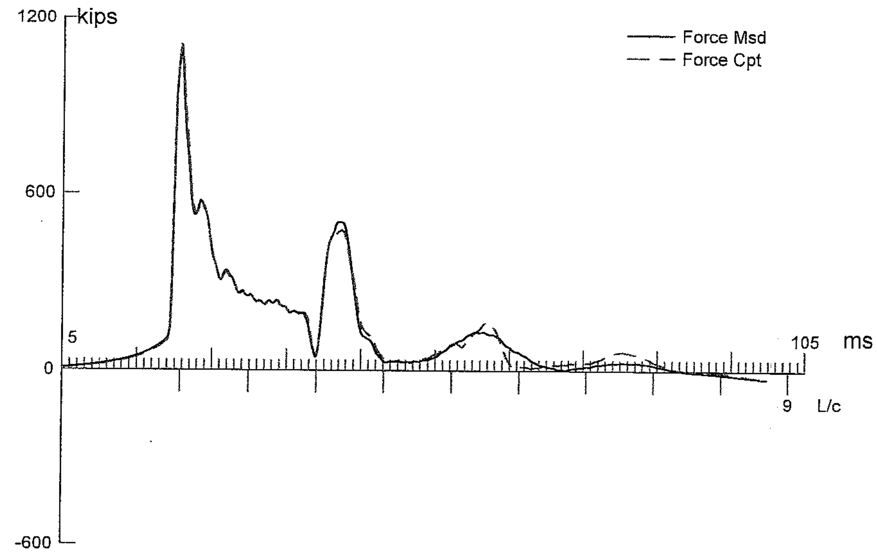
Time Summary

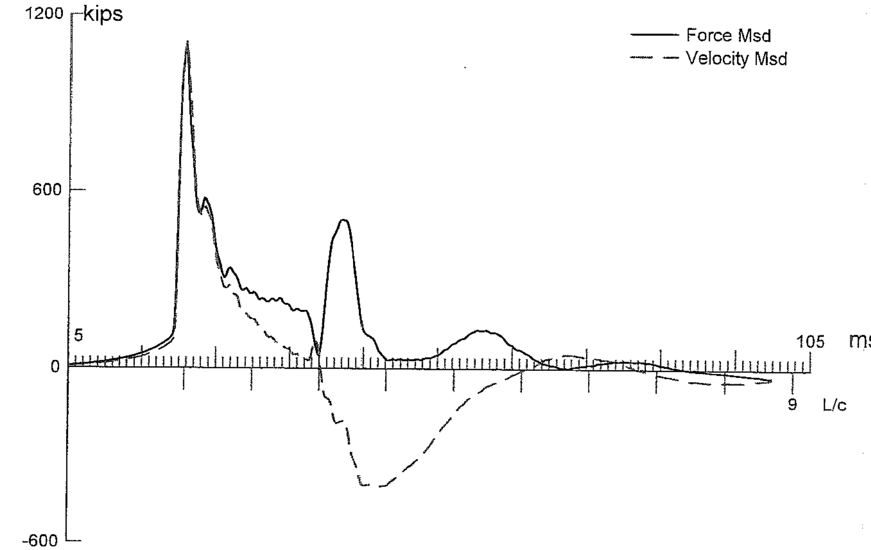
Drive	14 minutes 26 seconds	2:24:46 PM - 2:39:12 PM (4/21/2010) BN 1 - 622
Stop	19 hours 46 minutes 7 seconds	2:39:12 PM - 10:25:19 AM
Drive	4 minutes	10:25:19 AM - 10:29:19 AM BN 630 - 725
Stop	30 minutes 13 seconds	10:29:19 AM - 10:59:32 AM
Drive	12 minutes 6 seconds	10:59:32 AM - 11:11:38 AM BN 728 - 1054

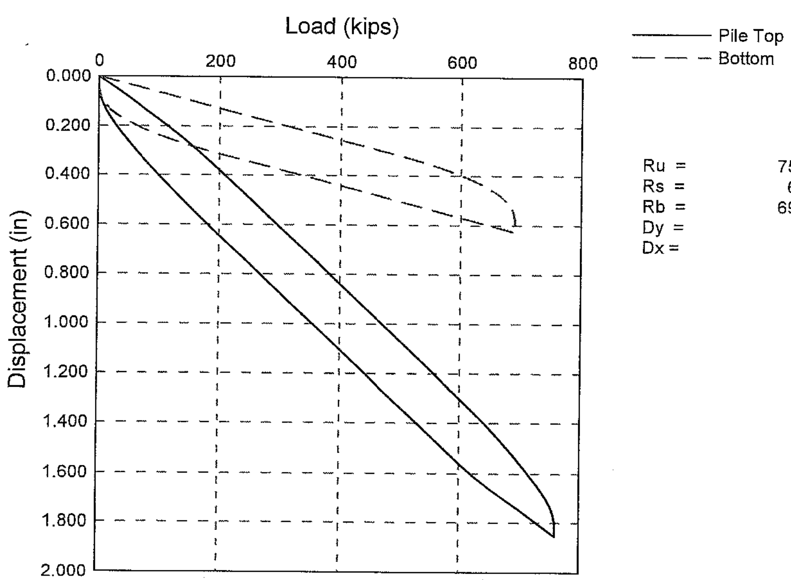
Total time [20:46:52] = (Driving [0:30:32] + Stop [20:16:20])

Appendix C

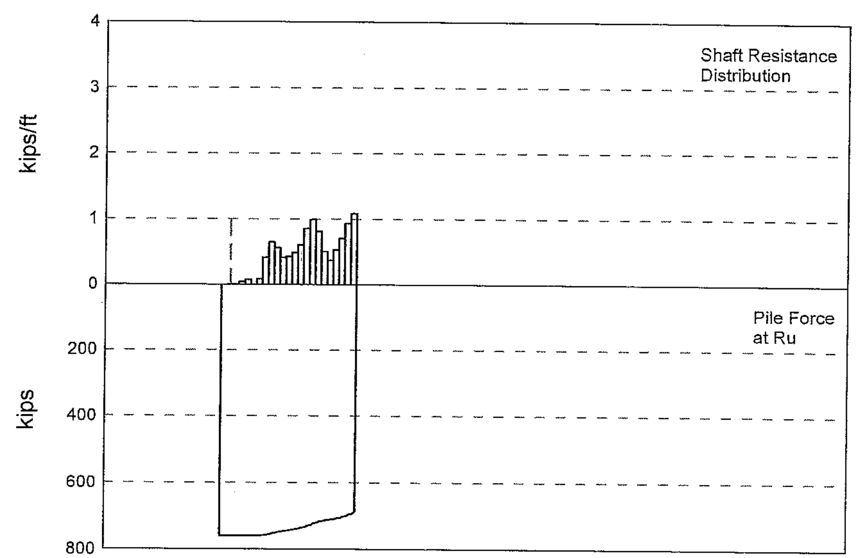
Results of CAPWAP Analysis







Ru =	759.8 kips
Rs =	69.8 kips
Rb =	690.0 kips
Dy =	1.80 in
Dx =	1.85 in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 759.8; along Shaft 69.8; at Toe 690.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				759.8				
1	13.3	1.6	0.0	759.8	0.0	0.00	0.00	0.000
2	20.0	8.3	0.0	759.8	0.0	0.00	0.00	0.000
3	26.6	14.9	0.3	759.5	0.3	0.05	0.01	0.177
4	33.3	21.6	0.5	759.0	0.8	0.08	0.01	0.177
5	39.9	28.2	0.0	759.0	0.8	0.00	0.00	0.000
6	46.6	34.9	0.6	758.4	1.4	0.09	0.01	0.177
7	53.2	41.5	2.7	755.7	4.1	0.41	0.06	0.177
8	59.9	48.2	4.3	751.4	8.4	0.65	0.10	0.177
9	66.5	54.9	3.7	747.7	12.1	0.56	0.09	0.177
10	73.2	61.5	2.7	745.0	14.8	0.41	0.06	0.177
11	79.8	68.2	2.8	742.2	17.6	0.42	0.07	0.177
12	86.5	74.8	3.2	739.0	20.8	0.48	0.08	0.177
13	93.1	81.5	4.0	735.0	24.8	0.60	0.10	0.177
14	99.8	88.1	5.7	729.3	30.5	0.86	0.14	0.177
15	106.4	94.8	6.6	722.7	37.1	0.99	0.16	0.177
16	113.1	101.4	5.4	717.3	42.5	0.81	0.13	0.177
17	119.7	108.1	3.3	714.0	45.8	0.50	0.08	0.177
18	126.4	114.7	2.4	711.6	48.2	0.36	0.06	0.177
19	133.0	121.4	3.5	708.1	51.7	0.53	0.08	0.177
20	139.7	128.0	4.7	703.4	56.4	0.71	0.11	0.177
21	146.3	134.7	6.2	697.2	62.6	0.93	0.15	0.177
22	153.0	141.3	7.2	690.0	69.8	1.08	0.17	0.177
Avg. Shaft			3.2			0.49	0.08	0.177
Toe			690.0				219.63	0.060

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.100	0.440
Case Damping Factor		0.187	0.628
Unloading Quake	(% of loading quake)	30	50
Reloading Level	(% of Ru)	100	100

max. Top Comp. Stress = 30.0 ksi (T= 21.2 ms, max= 1.012 x Top)
max. Comp. Stress = 30.3 ksi (Z= 53.2 ft, T= 24.1 ms)
max. Tens. Stress = -3.34 ksi (Z= 139.7 ft, T= 57.2 ms)
max. Energy (EMX) = 49.5 kip-ft; max. Measured Top Displ. (DMX)= 1.11 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	1106.1	-28.9	30.0	-0.78	49.53	16.6	1.109
2	6.7	1106.4	-30.8	30.0	-0.83	49.50	16.6	1.104
5	16.6	1107.6	-33.8	30.0	-0.92	49.31	16.6	1.083
8	26.6	1110.0	-35.7	30.1	-0.97	49.06	16.5	1.059
11	36.6	1109.1	-37.7	30.0	-1.02	48.61	16.4	1.033
14	46.6	1114.7	-44.5	30.2	-1.21	48.24	16.3	1.002
17	56.5	1111.9	-53.8	30.1	-1.46	47.13	16.2	0.968
20	66.5	1105.3	-59.3	29.9	-1.61	45.83	16.0	0.932
23	76.5	1089.0	-61.4	29.5	-1.66	44.13	15.8	0.894
26	86.5	1089.0	-86.6	29.5	-2.35	42.95	15.6	0.852
29	96.5	1075.5	-101.9	29.1	-2.76	40.98	15.4	0.807
32	106.4	1070.2	-114.0	29.0	-3.09	39.09	15.1	0.757
35	116.4	1037.2	-109.9	28.1	-2.98	36.02	14.9	0.699
38	126.4	1034.7	-114.2	28.0	-3.09	34.25	14.8	0.641
39	129.7	1029.9	-114.8	27.9	-3.11	33.43	14.7	0.620
40	133.0	1034.4	-120.8	28.0	-3.27	32.88	14.6	0.598
41	136.4	1026.7	-118.6	27.8	-3.21	31.89	14.5	0.576
42	139.7	1032.0	-123.5	27.9	-3.34	31.29	14.4	0.554
43	143.0	1005.2	-120.3	27.2	-3.26	30.14	14.7	0.531
44	146.3	938.4	-119.1	25.4	-3.22	29.48	15.7	0.507
45	149.7	886.7	-109.3	24.0	-2.96	28.14	16.0	0.483
46	153.0	946.0	-108.4	25.6	-2.94	28.02	15.3	0.458
Absolute	53.2			30.3			(T =	24.1 ms)
	139.7				-3.34		(T =	57.2 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1105.1	992.7	880.2	767.8	655.4	542.9	430.5	318.1	205.7	93.2
RX	1113.6	1006.5	899.3	846.1	823.1	800.1	777.0	754.0	730.9	707.9
RU	1105.1	992.7	880.2	767.8	655.4	542.9	430.5	318.1	205.7	93.2

RAU = 481.7 (kips); RA2 = 688.6 (kips)

Current CAPWAP Ru = 759.8 (kips); Corresponding J(RP)= 0.31; J(RX) = 0.67

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
16.91	20.98	1113.8	1115.5	1115.5	1.113	0.056	0.056	49.8	1022.6

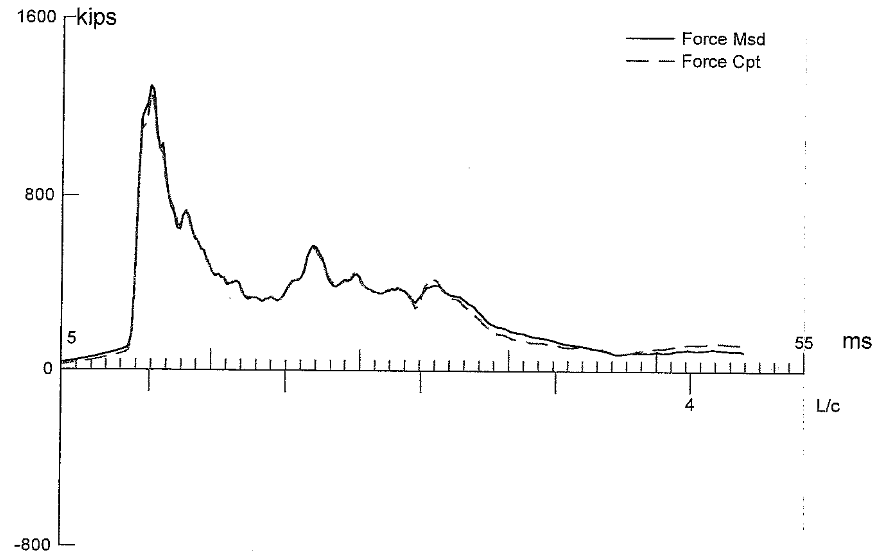
PILE PROFILE AND PILE MODEL

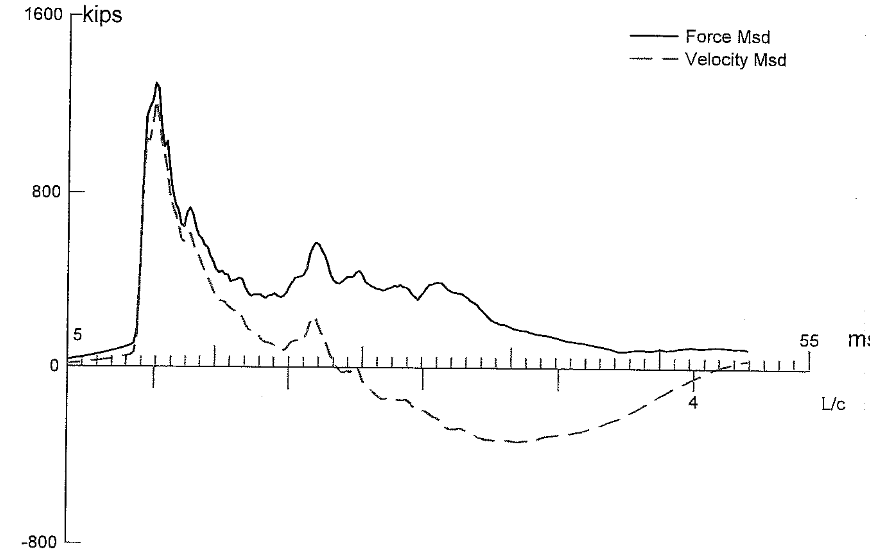
Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	36.91	29992.2	492.000	6.280
153.00	36.91	29992.2	492.000	6.280

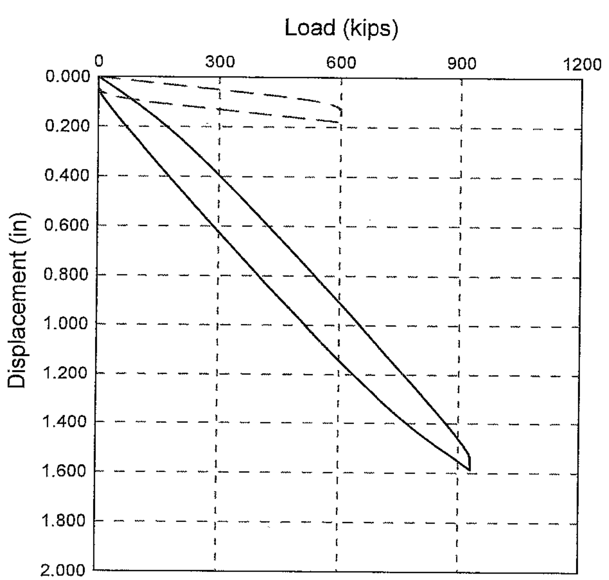
Toe Area 3.142 ft^2

Top Segment Length 3.33 ft, Top Impedance 65.89 kips/ft/s

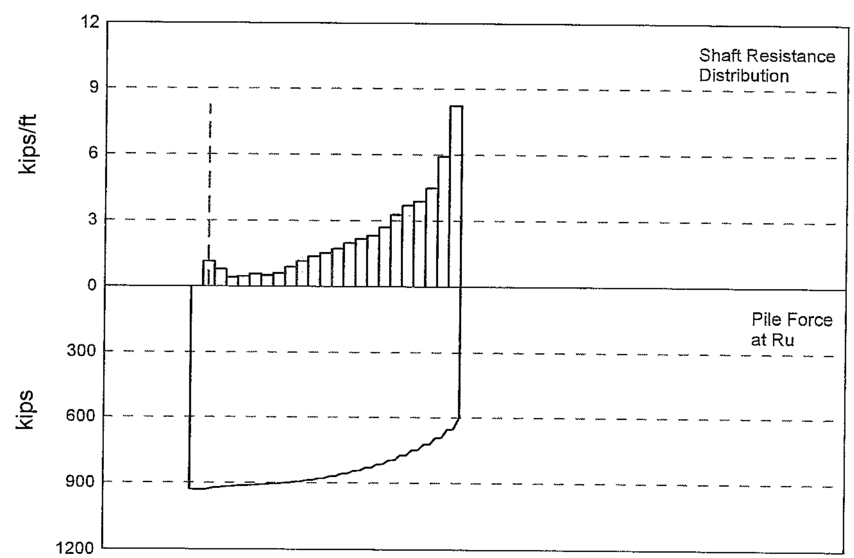
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 16807.9 ft/s, 2L/c 18.2 ms







Ru =	930.2 kips
Rs =	330.2 kips
Rb =	600.0 kips
Dy =	1.54 in
Dx =	1.59 in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 930.2; along Shaft 330.2; at Toe 600.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft	Quake in
				930.2					
1	13.3	1.6	7.5	922.7	7.5	4.67	0.74	0.360	0.100
2	20.0	8.3	5.2	917.5	12.7	0.78	0.12	0.360	0.100
3	26.6	14.9	2.8	914.7	15.5	0.42	0.07	0.360	0.100
4	33.3	21.6	3.1	911.6	18.6	0.47	0.07	0.360	0.100
5	39.9	28.2	3.8	907.8	22.4	0.57	0.09	0.360	0.100
6	46.6	34.9	3.3	904.5	25.7	0.50	0.08	0.360	0.100
7	53.2	41.5	3.9	900.6	29.6	0.59	0.09	0.360	0.100
8	59.9	48.2	5.8	894.8	35.4	0.87	0.14	0.360	0.100
9	66.5	54.8	7.6	887.2	43.0	1.14	0.18	0.360	0.100
10	73.2	61.5	9.0	878.2	52.0	1.35	0.22	0.360	0.100
11	79.8	68.1	10.0	868.2	62.0	1.50	0.24	0.360	0.100
12	86.5	74.8	11.3	856.9	73.3	1.70	0.27	0.360	0.100
13	93.1	81.4	13.1	843.8	86.4	1.97	0.31	0.360	0.100
14	99.8	88.1	14.4	829.4	100.8	2.16	0.34	0.360	0.100
15	106.4	94.7	15.4	814.0	116.2	2.32	0.37	0.360	0.100
16	113.1	101.4	17.9	796.1	134.1	2.69	0.43	0.360	0.100
17	119.7	108.0	21.8	774.3	155.9	3.28	0.52	0.360	0.100
18	126.4	114.7	24.6	749.7	180.5	3.70	0.59	0.360	0.100
19	133.0	121.3	25.9	723.8	206.4	3.89	0.62	0.360	0.100
20	139.7	128.0	29.8	694.0	236.2	4.48	0.71	0.360	0.097
21	146.3	134.6	39.3	654.7	275.5	5.91	0.94	0.360	0.087
22	153.0	141.3	54.7	600.0	330.2	8.22	1.31	0.360	0.068
Avg. Shaft			15.0			2.34	0.37	0.360	0.093
Toe			600.0				190.99	0.070	0.100

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.804	0.637
Damping Type			Smith
Unloading Quake	(% of loading quake)	30	100
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	10	
Soil Plug Weight	(kips)		0.35

max. Top Comp. Stress = 34.2 ksi (T= 11.3 ms, max= 1.027 x Top)
max. Comp. Stress = 35.2 ksi (Z= 13.3 ft, T= 11.9 ms)
max. Tens. Stress = -0.57 ksi (Z= 139.7 ft, T= 51.1 ms)
max. Energy (EMX) = 59.8 kip-ft; max. Measured Top Displ. (DMX)= 0.96 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	1264.2	0.0	34.2	0.00	59.75	18.9	0.975
2	6.7	1278.5	0.0	34.6	0.00	59.48	18.6	0.960
5	16.6	1247.4	0.0	33.8	0.00	56.06	18.2	0.912
8	26.6	1225.6	0.0	33.2	0.00	53.35	17.8	0.862
11	36.6	1198.8	0.0	32.5	0.00	50.40	17.5	0.807
14	46.6	1191.4	0.0	32.3	0.00	48.05	17.1	0.751
17	56.5	1165.1	0.0	31.6	0.00	44.93	16.7	0.695
20	66.5	1166.3	0.0	31.6	0.00	42.03	16.0	0.639
23	76.5	1098.1	0.0	29.7	0.00	37.47	15.3	0.590
26	86.5	1090.6	-3.7	29.5	-0.10	34.52	14.4	0.539
29	96.5	1000.9	-8.1	27.1	-0.22	29.31	13.5	0.486
32	106.4	989.8	-16.4	26.8	-0.44	26.19	12.4	0.435
35	116.4	887.4	-17.0	24.0	-0.46	21.01	11.3	0.379
38	126.4	870.9	-20.8	23.6	-0.56	17.39	10.0	0.316
39	129.7	781.6	-18.1	21.2	-0.49	14.79	9.6	0.293
40	133.0	817.5	-21.1	22.1	-0.57	14.10	9.1	0.269
41	136.4	732.5	-18.7	19.8	-0.51	11.70	8.8	0.244
42	139.7	774.1	-21.2	21.0	-0.57	10.94	8.2	0.219
43	143.0	686.2	-17.4	18.6	-0.47	8.74	7.8	0.194
44	146.3	699.5	-18.4	18.9	-0.50	8.03	8.0	0.169
45	149.7	697.8	-12.0	18.9	-0.32	5.91	8.3	0.146
46	153.0	759.8	-12.3	20.6	-0.33	4.54	7.7	0.123
Absolute	13.3			35.2			(T =	11.9 ms)
	139.7				-0.57		(T =	51.1 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1523.5	1425.0	1326.6	1228.1	1129.7	1031.2	932.8	834.3	735.9	637.4
RX	1523.5	1425.0	1326.6	1228.1	1129.7	1031.2	934.8	841.7	748.7	679.8
RU	1603.3	1512.8	1422.3	1331.9	1241.4	1150.9	1060.5	970.0	879.5	789.1

RAU = 498.4 (kips); RA2 = 706.2 (kips)

Current CAPWAP Ru = 930.2 (kips); Corresponding J(RP)= 0.60; J(RX) = 0.60

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
18.44	11.08	1214.8	1293.1	1293.1	0.963	0.049	0.050	59.9	1418.9

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	36.91	29992.2	492.000	6.283
153.00	36.91	29992.2	492.000	6.283

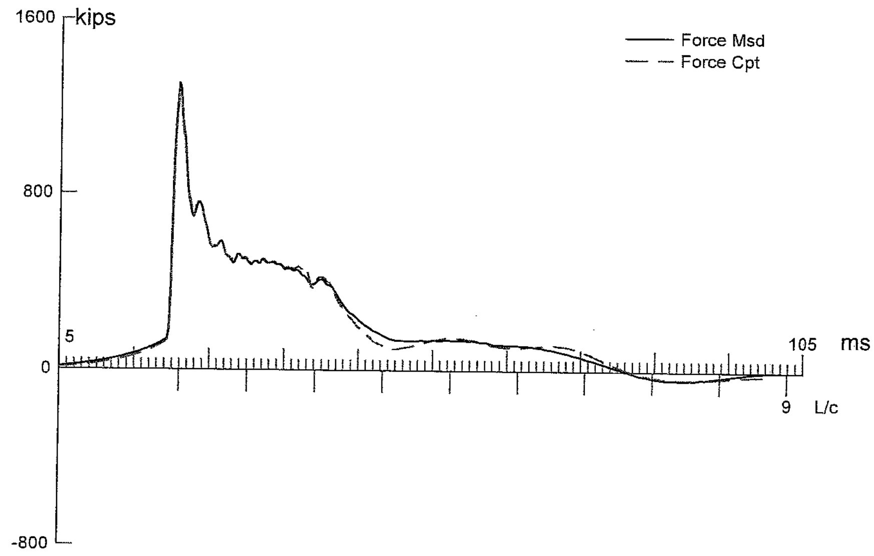
Toe Area 3.142 ft^2

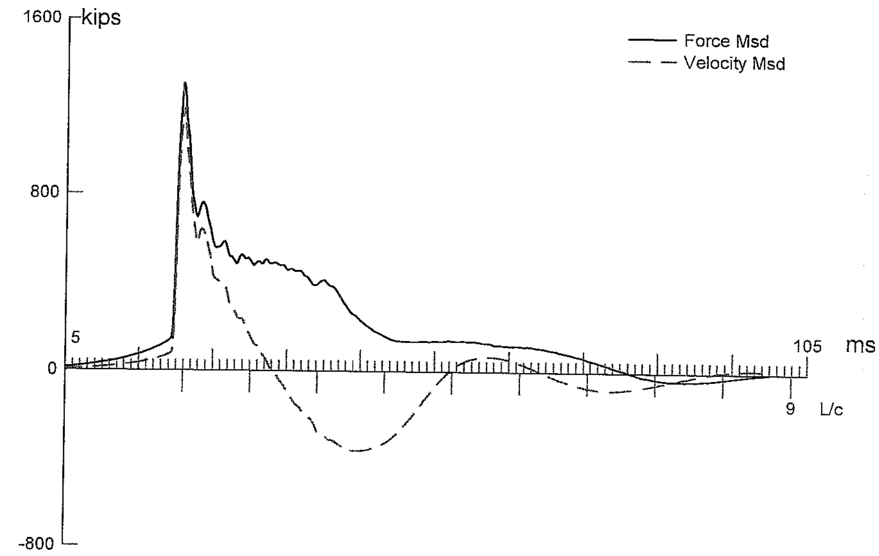
Top Segment Length 3.33 ft, Top Impedance 65.89 kips/ft/s

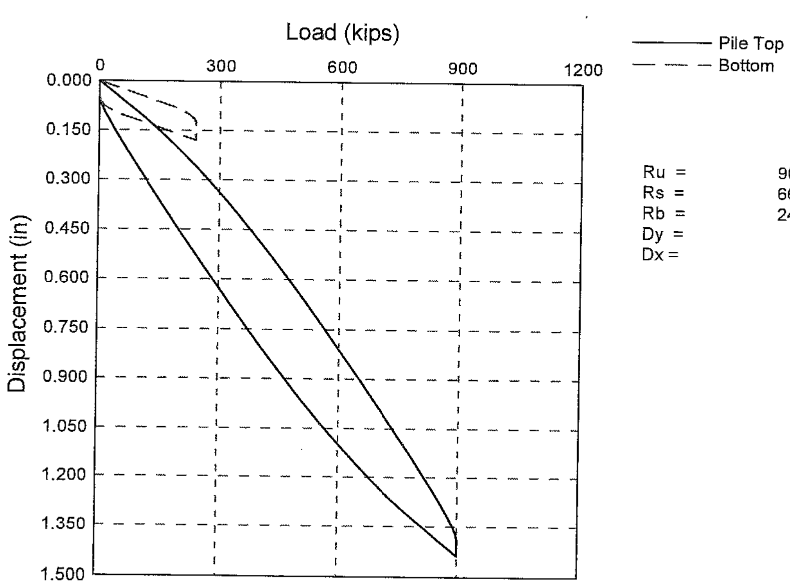
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 16807.9 ft/s, 2L/c 18.2 ms

GCC, SR520; Pile: P1 2nd RESTRIKE; PP24x0..50, Closed-End, D62-22; Blow: 5 (Test: 26-Apr-2010 16:46:)
Robert Miner Dynamic Testing, Inc.

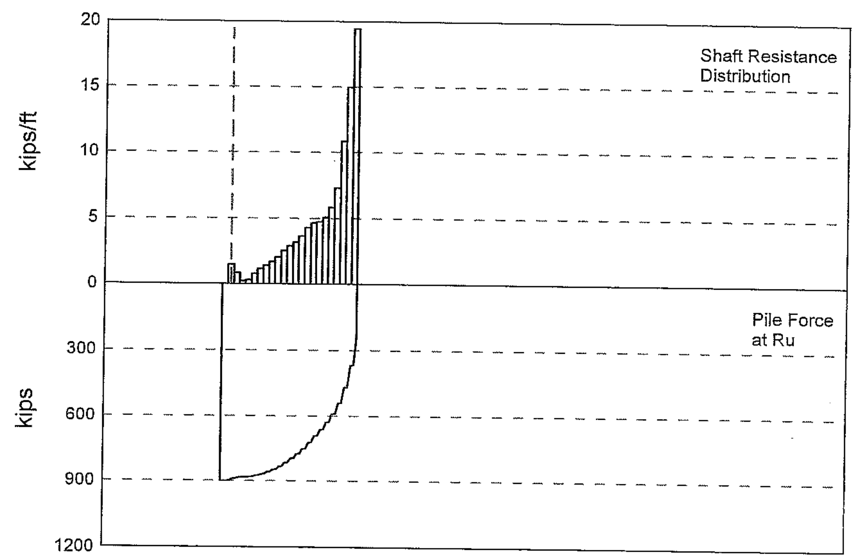
11-May-2010
CAPWAP(R) 2006-3







Ru = 900.1 kips
Rs = 660.1 kips
Rb = 240.0 kips
Dy = 1.39 in
Dx = 1.44 in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 900.1; along Shaft 660.1; at Toe 240.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				900.1				
1	13.3	1.8	9.8	890.3	9.8	5.43	0.86	0.180
2	20.0	8.5	5.6	884.7	15.4	0.84	0.13	0.180
3	26.6	15.1	1.4	883.3	16.8	0.21	0.03	0.180
4	33.3	21.8	1.9	881.4	18.7	0.29	0.05	0.180
5	39.9	28.4	5.3	876.1	24.0	0.80	0.13	0.180
6	46.6	35.1	7.7	868.4	31.7	1.16	0.18	0.180
7	53.2	41.7	9.4	859.0	41.1	1.41	0.22	0.180
8	59.9	48.4	11.2	847.8	52.3	1.68	0.27	0.180
9	66.5	55.0	13.5	834.3	65.8	2.03	0.32	0.180
10	73.2	61.7	16.7	817.6	82.5	2.51	0.40	0.180
11	79.8	68.3	19.3	798.3	101.8	2.90	0.46	0.180
12	86.5	75.0	21.0	777.3	122.8	3.16	0.50	0.180
13	93.1	81.6	24.2	753.1	147.0	3.64	0.58	0.180
14	99.8	88.3	28.5	724.6	175.5	4.28	0.68	0.180
15	106.4	94.9	30.9	693.7	206.4	4.65	0.74	0.180
16	113.1	101.6	31.6	662.1	238.0	4.75	0.76	0.180
17	119.7	108.2	33.8	628.3	271.8	5.08	0.81	0.180
18	126.4	114.9	38.7	589.6	310.5	5.82	0.93	0.180
19	133.0	121.5	48.4	541.2	358.9	7.28	1.16	0.180
20	139.7	128.2	72.1	469.1	431.0	10.84	1.73	0.180
21	146.3	134.8	99.6	369.5	530.6	14.97	2.38	0.180
22	153.0	141.5	129.5	240.0	660.1	19.47	3.10	0.180
Avg. Shaft			30.0			4.67	0.74	0.180
Toe			240.0				76.39	0.030

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.100	0.100
Case Damping Factor		1.803	0.109
Damping Type			Smith
Unloading Quake	(% of loading quake)	90	100
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	21	

max. Top Comp. Stress = 34.5 ksi (T= 21.2 ms, max= 1.017 x Top)
max. Comp. Stress = 35.1 ksi (Z= 13.3 ft, T= 21.8 ms)
max. Tens. Stress = -1.28 ksi (Z= 53.2 ft, T= 90.0 ms)
max. Energy (EMX) = 68.1 kip-ft; max. Measured Top Displ. (DMX)= 1.11 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	1274.1	-36.5	34.5	-0.99	68.11	18.4	1.120
2	6.7	1281.2	-38.2	34.7	-1.04	67.72	18.3	1.102
5	16.6	1258.5	-39.9	34.1	-1.08	64.24	17.9	1.046
8	26.6	1242.9	-42.3	33.7	-1.15	61.65	17.7	0.987
11	36.6	1242.2	-45.5	33.6	-1.23	59.52	17.5	0.925
14	46.6	1247.1	-47.4	33.8	-1.28	56.94	17.0	0.862
17	56.5	1209.4	-45.8	32.8	-1.24	52.29	16.5	0.796
20	66.5	1207.5	-45.5	32.7	-1.23	48.66	15.9	0.727
23	76.5	1140.9	-39.9	30.9	-1.08	42.60	15.1	0.659
26	86.5	1126.4	-37.1	30.5	-1.01	38.49	14.3	0.593
29	96.5	1031.3	-28.2	27.9	-0.76	32.02	13.3	0.527
32	106.4	1007.8	-23.3	27.3	-0.63	27.67	12.2	0.459
35	116.4	880.4	-10.7	23.8	-0.29	21.22	11.2	0.391
38	126.4	865.9	-4.9	23.5	-0.13	17.53	10.0	0.325
39	129.7	785.3	0.0	21.3	0.00	15.16	9.6	0.305
40	133.0	827.2	0.0	22.4	0.00	14.72	9.1	0.286
41	136.4	738.1	0.0	20.0	0.00	12.36	8.6	0.267
42	139.7	786.7	0.0	21.3	0.00	11.95	8.0	0.249
43	143.0	643.2	0.0	17.4	0.00	9.27	7.6	0.232
44	146.3	603.6	0.0	16.3	0.00	8.97	8.6	0.216
45	149.7	458.0	0.0	12.4	0.00	6.06	8.7	0.203
46	153.0	516.8	0.0	14.0	0.00	3.11	7.9	0.189
Absolute	13.3			35.1			(T =	21.8 ms)
	53.2				-1.28		(T =	90.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1601.3	1507.6	1413.8	1320.1	1226.3	1132.6	1038.8	945.1	851.3	757.6
RX	1601.3	1507.6	1413.8	1320.1	1226.3	1132.6	1038.8	945.1	854.1	769.2
RU	1651.7	1563.0	1474.3	1385.6	1296.9	1208.2	1119.5	1030.8	942.1	853.4

RAU = 127.2 (kips); RA2 = 789.3 (kips)

Current CAPWAP Ru = 900.1 (kips); Corresponding J(RP)= 0.75; J(RX) = 0.75

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
18.56	21.17	1223.1	1315.7	1325.4	1.107	0.050	0.050	68.6	1422.1

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	36.91	29992.2	492.000	6.283
153.00	36.91	29992.2	492.000	6.283

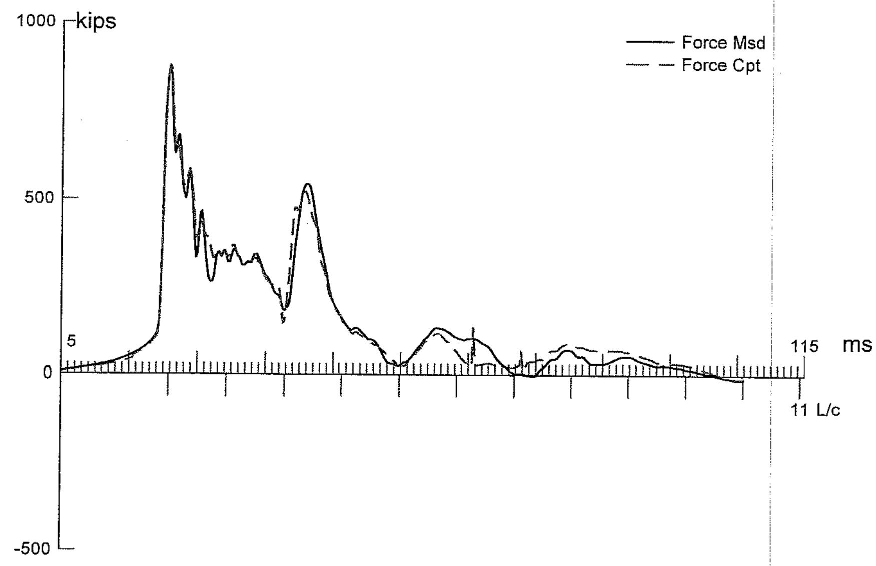
Toe Area 3.142 ft^2

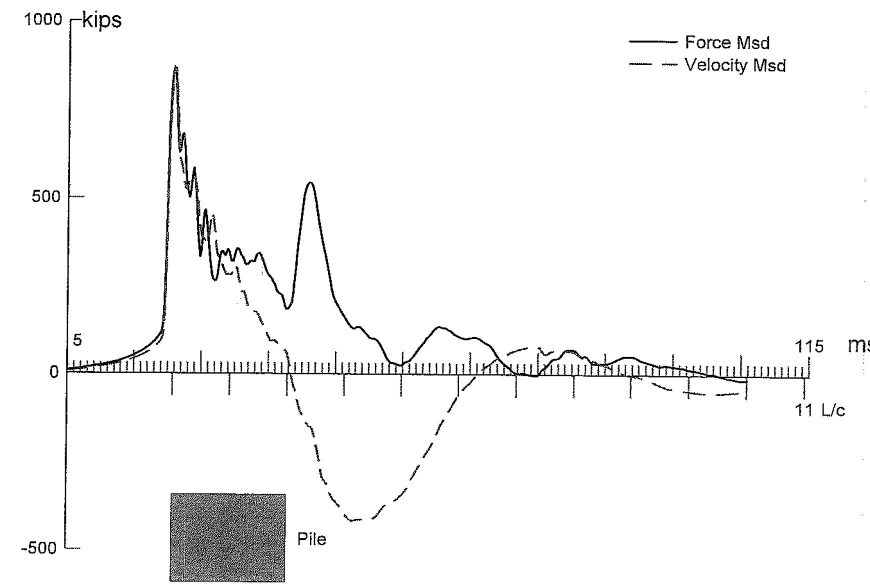
Top Segment Length 3.33 ft, Top Impedance 65.89 kips/ft/s

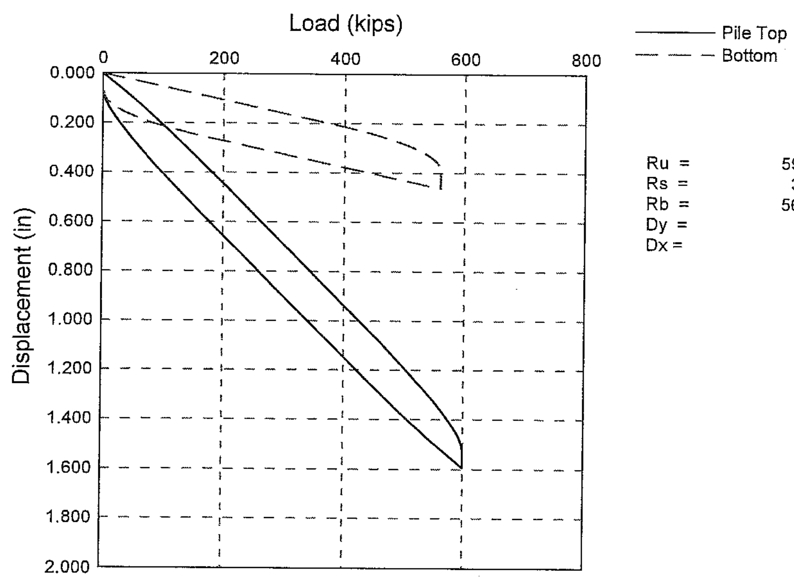
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 16807.9 ft/s, 2L/c 18.2 ms

GCC, SR520; Pile: P2, End Drive; PP24"x0.401" CLOSED END; Blow: 1385 (Test: 15-Apr-2010 13:19:)
Robert Miner Dynamic Testing, Inc.

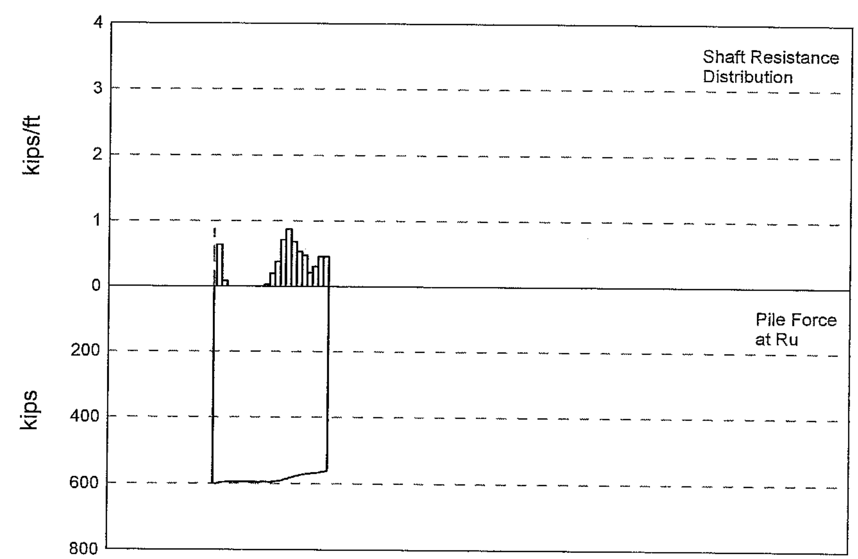
11-May-2010
CAPWAP(R) 2006-3







Ru = 599.8 kips
Rs = 39.8 kips
Rb = 560.0 kips
Dy = 1.52 in
Dx = 1.59 in



GCC, SR520; Pile: P2, End Drive | Test: 15-Apr-2010 13:19:
PP24"x0.401" CLOSED END; Blow: 1385 | CAPWAP(R) 2006-3
Robert Miner Dynamic Testing, Inc. | OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 599.8; along Shaft 39.8; at Toe 560.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				599.8				
1	10.0	9.0	4.2	595.6	4.2	0.47	0.07	0.300
2	16.6	15.6	0.6	595.0	4.8	0.09	0.01	0.300
3	23.3	22.3	0.0	595.0	4.8	0.00	0.00	0.000
4	29.9	28.9	0.0	595.0	4.8	0.00	0.00	0.000
5	36.6	35.6	0.0	595.0	4.8	0.00	0.00	0.000
6	43.2	42.2	0.0	595.0	4.8	0.00	0.00	0.000
7	49.9	48.9	0.0	595.0	4.8	0.00	0.00	0.000
8	56.5	55.5	0.0	595.0	4.8	0.00	0.00	0.000
9	63.2	62.2	0.0	595.0	4.8	0.00	0.00	0.000
10	69.8	68.8	0.2	594.8	5.0	0.03	0.00	0.300
11	76.5	75.5	1.3	593.5	6.3	0.20	0.03	0.300
12	83.1	82.1	2.5	591.0	8.8	0.38	0.06	0.300
13	89.8	88.8	4.7	586.3	13.5	0.71	0.11	0.300
14	96.4	95.4	5.8	580.5	19.3	0.87	0.14	0.300
15	103.1	102.1	4.5	576.0	23.8	0.68	0.11	0.300
16	109.7	108.7	3.5	572.5	27.3	0.53	0.08	0.300
17	116.4	115.4	3.1	569.4	30.4	0.47	0.07	0.300
18	123.0	122.0	1.4	568.0	31.8	0.21	0.03	0.300
19	129.7	128.7	2.0	566.0	33.8	0.30	0.05	0.300
20	136.3	135.3	3.0	563.0	36.8	0.45	0.07	0.300
21	143.0	142.0	3.0	560.0	39.8	0.45	0.07	0.300
Avg. Shaft			1.9			0.28	0.04	0.300
Toe			560.0				178.25	0.110

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.100	0.300
Case Damping Factor		0.225	1.161
Damping Type			Smith
Unloading Quake	(% of loading quake)	30	81
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	30	
Soil Plug Weight	(kips)		0.40

max. Top Comp. Stress = 29.7 ksi (T= 21.2 ms, max= 1.005 x Top)
max. Comp. Stress = 29.8 ksi (Z= 10.0 ft, T= 21.6 ms)
max. Tens. Stress = -5.94 ksi (Z= 96.4 ft, T= 60.3 ms)
max. Energy (EMX) = 56.8 kip-ft; max. Measured Top Displ. (DMX)= 1.51 in

GCC, SR520; Pile: P2, End Drive
PP24"x0.401" CLOSED END; Blow: 1385
Robert Miner Dynamic Testing, Inc.

Test: 15-Apr-2010 13:19:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	882.5	-11.0	29.7	-0.37	56.84	16.2	1.468
2	6.7	885.6	-11.4	29.8	-0.38	56.71	16.1	1.457
5	16.6	863.8	-11.6	29.0	-0.39	54.55	16.0	1.417
8	26.6	856.4	-11.1	28.8	-0.37	53.65	16.1	1.371
11	36.6	861.2	-20.4	29.0	-0.69	52.73	16.0	1.317
14	46.6	865.2	-49.9	29.1	-1.68	51.39	15.8	1.251
17	56.5	868.8	-82.7	29.2	-2.78	50.01	15.7	1.185
20	66.5	872.8	-118.3	29.4	-3.98	48.36	15.6	1.112
23	76.5	880.5	-149.5	29.6	-5.03	46.13	15.4	1.028
26	86.5	876.3	-169.2	29.5	-5.69	42.97	15.1	0.936
29	96.4	870.6	-176.7	29.3	-5.94	39.59	14.7	0.853
32	106.4	830.8	-168.9	27.9	-5.68	35.07	14.4	0.772
35	116.4	824.6	-165.7	27.7	-5.57	32.46	14.1	0.695
36	119.7	811.7	-161.8	27.3	-5.44	31.19	14.0	0.669
37	123.0	815.7	-161.8	27.4	-5.44	30.56	13.9	0.643
38	126.4	820.7	-160.8	27.6	-5.41	29.70	13.9	0.617
39	129.7	835.4	-161.5	28.1	-5.43	29.07	13.8	0.592
40	133.0	822.7	-159.9	27.7	-5.38	28.14	13.5	0.566
41	136.3	808.2	-160.2	27.2	-5.39	27.53	14.3	0.540
42	139.7	850.2	-157.1	28.6	-5.28	26.46	15.1	0.514
43	143.0	876.9	-156.6	29.5	-5.26	26.24	14.4	0.489
Absolute	10.0			29.8			(T =	21.6 ms)
	96.4				-5.94		(T =	60.3 ms)

Analysis: 11-May-2010

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	948.4	867.5	786.5	705.6	624.7	543.8	462.8	381.9	301.0	220.0
RX	948.4	898.4	874.0	850.0	828.0	806.1	784.1	762.1	740.1	718.2
RU	948.4	867.5	786.5	705.6	624.7	543.8	462.8	381.9	301.0	220.0

RAU = 538.6 (kips); RA2 = 597.2 (kips)

Current CAPWAP Ru = 599.8 (kips); RMX requires J > 0.9;

Check with PDA-W; RA2 may be a better Case Method

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
16.60	20.97	880.7	877.0	877.0	1.507	0.074	0.074	57.0	864.5

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	29.73	29992.2	492.000	6.283
143.00	29.73	29992.2	492.000	6.283

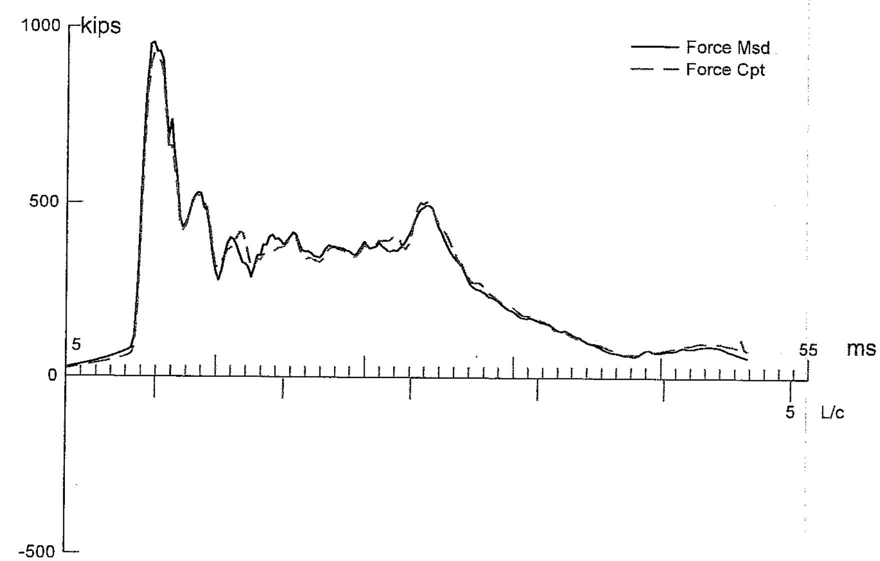
Toe Area 3.142 ft^2

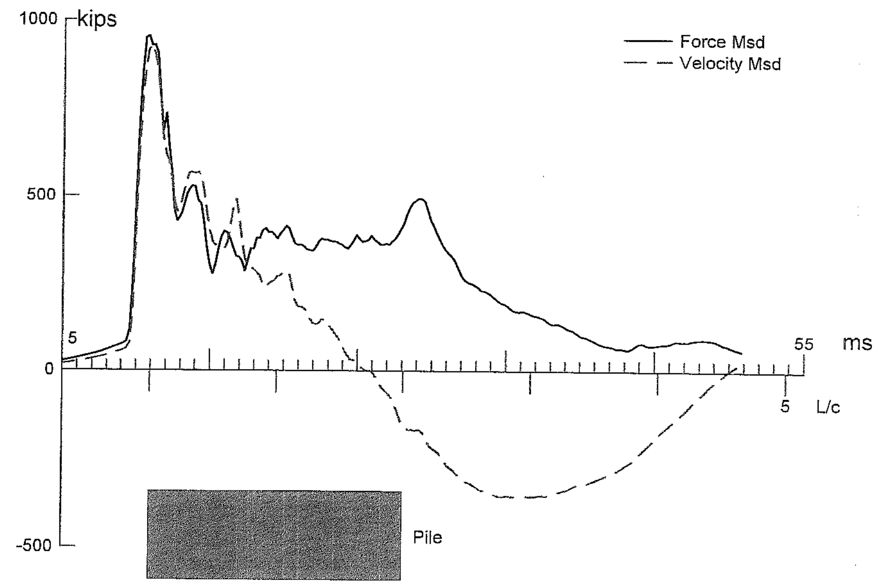
Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Tension Slack	Tension Eff.	Compression Slack	Compression Eff.	Perim.
	ft	kips/ft/s	%	in		in		ft
1	3.33	53.06	0.00	0.000	0.000	0.000	0.000	6.283
16	53.21	53.06	0.00	0.000	0.000	-0.100	0.000	6.283
17	56.53	53.06	0.00	0.000	0.000	0.000	0.000	6.283
25	83.14	53.06	0.00	0.000	0.000	-0.010	0.900	6.283
26	86.47	53.06	0.00	0.000	0.000	0.000	0.000	6.283
43	143.00	53.06	0.00	0.000	0.000	0.000	0.000	6.283

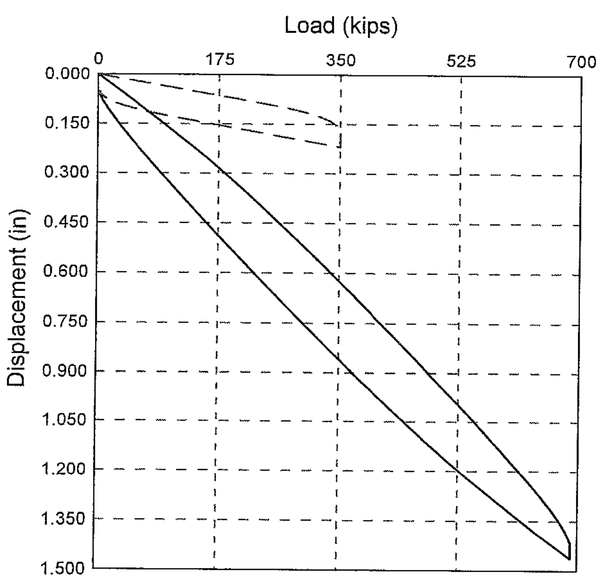
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 16807.9 ft/s, 2L/c 17.0 ms

KIEWIT GENERAL; Pile: PILE 2 1ST RESTRIKE; PP24x0.401", CLOSED-END, D62-22; Blow: 10 (Test: 19-Apr-2010 09:29:)
Robert Miner Dynamic Testing, Inc.

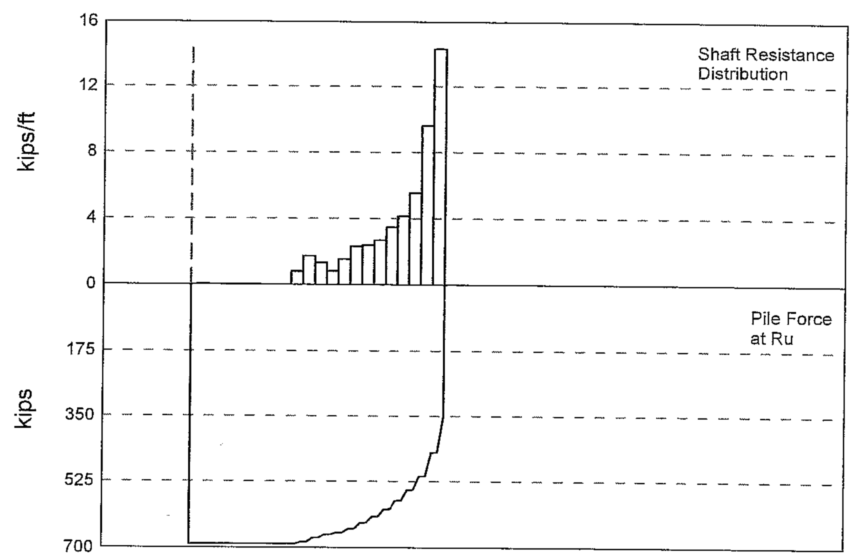
11-May-2010
CAPWAP(R) 2006-3







Ru =	690.0 kips
Rs =	340.0 kips
Rb =	350.0 kips
Dy =	1.41 in
Dx =	1.46 in



KIEWIT GENERAL; Pile: PILE 2 1ST RESTRIKE   Test: 19-Apr-2010 09:29:
PP24x0.401", CLOSED-END, D62-22; Blow: 10   CAPWAP(R) 2006-3
Robert Miner Dynamic Testing, Inc.   OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 690.0; along Shaft 340.0; at Toe 350.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				690.0				
1	10.0	8.0	0.3	689.7	0.3	0.04	0.01	0.188
2	16.7	14.7	0.0	689.7	0.3	0.00	0.00	0.000
3	23.4	21.4	0.0	689.7	0.3	0.00	0.00	0.000
4	30.1	28.1	0.0	689.7	0.3	0.00	0.00	0.000
5	36.8	34.8	0.0	689.7	0.3	0.00	0.00	0.000
6	43.5	41.5	0.0	689.7	0.3	0.00	0.00	0.000
7	50.2	48.2	0.0	689.7	0.3	0.00	0.00	0.000
8	56.9	54.9	0.1	689.6	0.4	0.01	0.00	0.188
9	63.6	61.6	5.3	684.3	5.7	0.79	0.13	0.188
10	70.3	68.3	11.5	672.8	17.2	1.72	0.27	0.188
11	77.0	75.0	8.8	664.0	26.0	1.31	0.21	0.188
12	83.7	81.7	5.5	658.5	31.5	0.82	0.13	0.188
13	90.4	88.4	10.2	648.3	41.7	1.52	0.24	0.188
14	97.1	95.1	15.5	632.8	57.2	2.31	0.37	0.188
15	103.8	101.8	16.0	616.8	73.2	2.39	0.38	0.188
16	110.5	108.5	18.0	598.8	91.2	2.69	0.43	0.188
17	117.2	115.2	23.3	575.5	114.5	3.48	0.55	0.188
18	123.9	121.9	27.7	547.8	142.2	4.14	0.66	0.188
19	130.6	128.6	37.1	510.7	179.3	5.54	0.88	0.188
20	137.3	135.3	64.5	446.2	243.8	9.63	1.53	0.188
21	144.0	142.0	96.2	350.0	340.0	14.36	2.29	0.188
Avg. Shaft			16.2			2.39	0.38	0.188
Toe			350.0				111.41	0.096

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.076	0.130
Case Damping Factor		1.206	0.633
Damping Type			Smith
Unloading Quake	(% of loading quake)	30	100
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	0	
Soil Plug Weight	(kips)		0.22

max. Top Comp. Stress = 31.1 ksi (T= 11.2 ms, max= 1.039 x Top)
max. Comp. Stress = 32.3 ksi (Z= 63.6 ft, T= 14.9 ms)
max. Tens. Stress = -4.03 ksi (Z= 103.8 ft, T= 49.0 ms)
max. Energy (EMX) = 56.7 kip-ft; max. Measured Top Displ. (DMX)= 1.31 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	924.0	0.0	31.1	0.00	56.75	17.9	1.338
2	6.7	922.3	0.0	31.0	0.00	56.45	17.9	1.320
5	16.7	913.9	0.0	30.7	0.00	55.41	18.0	1.265
8	26.8	904.9	0.0	30.4	0.00	54.26	18.2	1.204
11	36.8	926.5	-3.7	31.2	-0.12	52.89	17.7	1.138
14	46.9	935.6	-29.6	31.5	-1.00	51.40	17.5	1.068
17	56.9	943.0	-51.0	31.7	-1.72	49.93	17.3	0.999
20	67.0	948.0	-71.9	31.9	-2.42	46.83	16.8	0.869
23	77.0	924.7	-88.1	31.1	-2.96	42.20	16.3	0.780
26	87.1	895.2	-102.7	30.1	-3.45	37.59	15.8	0.687
29	97.1	899.9	-116.8	30.3	-3.93	33.98	15.0	0.604
32	107.2	832.4	-116.9	28.0	-3.93	27.80	14.1	0.518
35	117.2	834.3	-113.1	28.1	-3.80	23.80	12.9	0.434
36	120.6	784.5	-107.5	26.4	-3.62	21.18	12.5	0.409
37	123.9	801.4	-107.8	26.9	-3.62	20.57	11.9	0.384
38	127.3	757.7	-100.6	25.5	-3.38	18.04	11.6	0.360
39	130.6	793.6	-101.0	26.7	-3.40	17.51	10.8	0.336
40	134.0	714.7	-91.1	24.0	-3.06	14.84	10.0	0.313
41	137.3	708.0	-91.1	23.8	-3.06	14.36	10.3	0.290
42	140.7	628.2	-73.8	21.1	-2.48	10.98	10.3	0.270
43	144.0	714.8	-73.7	24.0	-2.48	7.51	9.5	0.250
Absolute	63.6			32.3			(T =	14.9 ms)
	103.8				-4.03		(T =	49.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1218.6	1155.2	1091.9	1028.5	965.2	901.8	838.4	775.1	711.7	648.4
RX	1227.0	1167.3	1107.6	1047.8	988.1	928.4	869.3	811.4	754.6	700.4
RU	1210.5	1146.4	1082.2	1018.0	953.9	889.7	825.6	761.4	697.3	633.1

RAU = 319.1 (kips); RA2 = 604.3 (kips)

Current CAPWAP Ru = 690.0 (kips); Corresponding J(RP)= 0.83; matches RX9 within 5%

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
17.58	11.16	932.8	919.3	971.5	1.311	0.050	0.050	56.7	1000.1

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	29.73	29992.2	492.000	6.283
144.00	29.73	29992.2	492.000	6.283

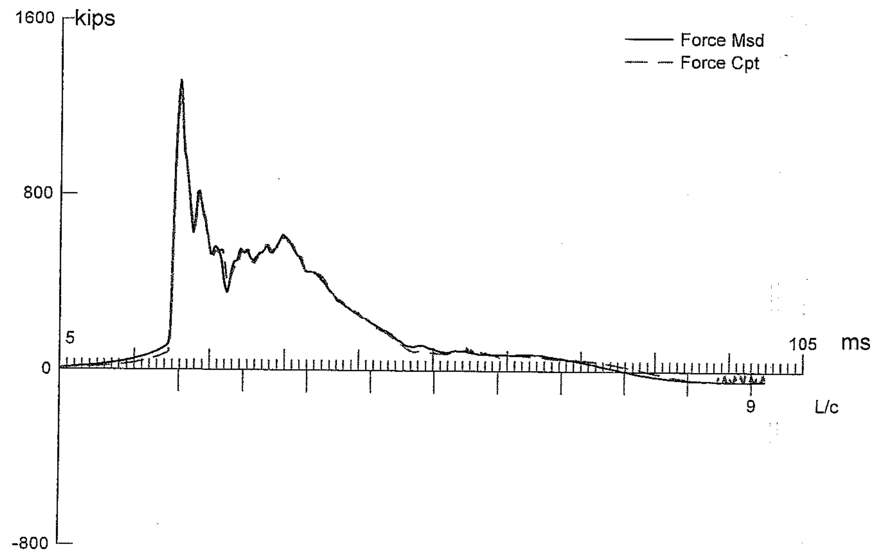
Toe Area 3.142 ft^2

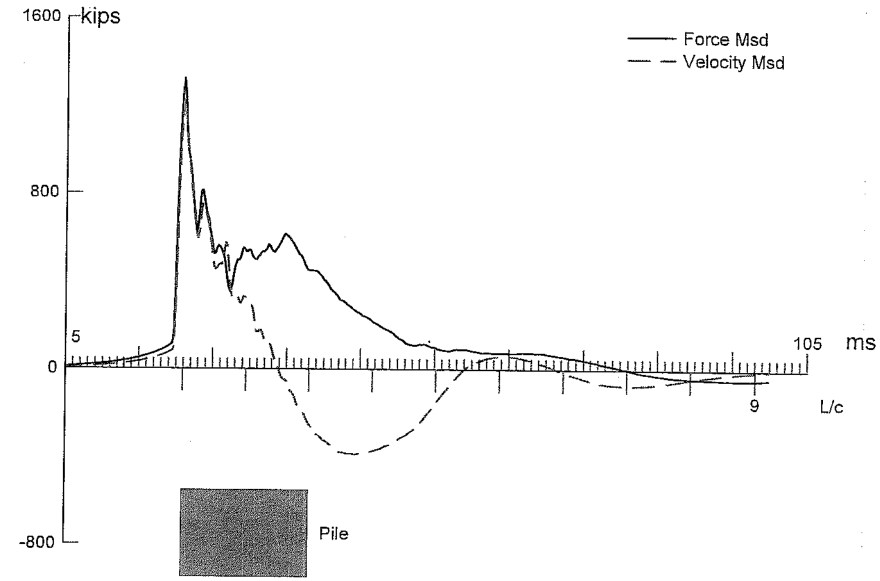
Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Tension Slack	Tension Eff.	Compression Slack	Compression Eff.	Perim.
	ft	kips/ft/s	%	in		in		ft
1	3.35	53.06	0.00	0.000	0.000	0.000	0.000	6.283
17	56.93	53.06	0.00	0.000	0.000	-0.050	0.700	6.283
18	60.28	53.06	0.00	0.000	0.000	0.000	0.000	6.283
43	144.00	53.06	0.00	0.000	0.000	0.000	0.000	6.283

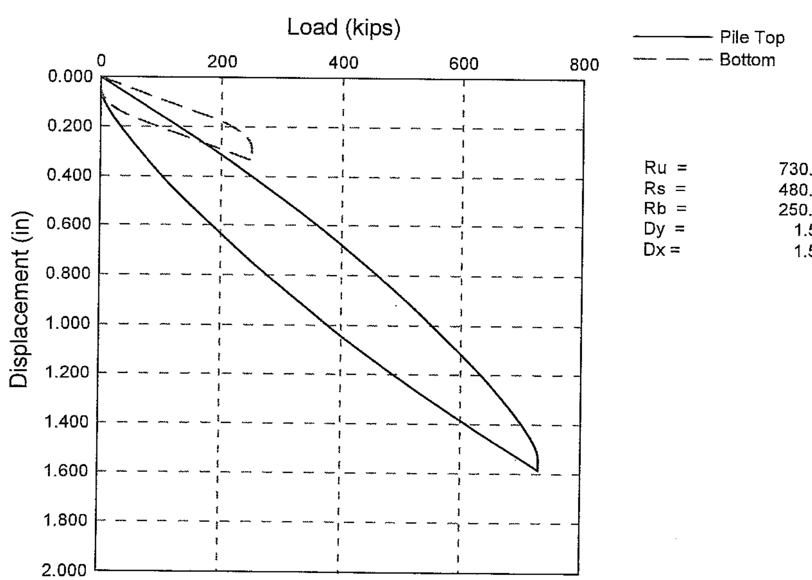
Pile Damping 1.0 %, Time Incr 0.199 ms, Wave Speed 16807.9 ft/s, 2L/c 17.1 ms

GCC, LOGYARD; Pile: P2 2nd RESTRIKE; PP24x0.401, D62-22; Blow: 54 (Test: 26-Apr-2010 16:33:)
Robert Miner Dynamic Testing, Inc.

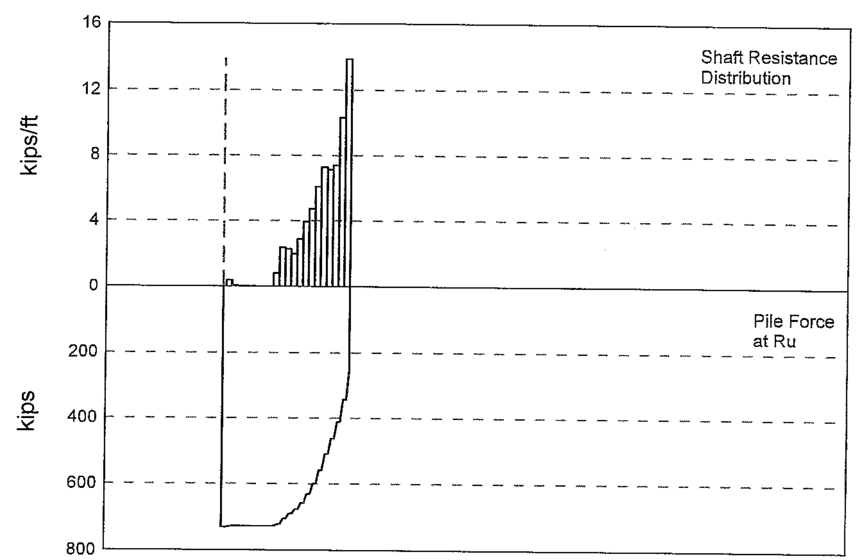
11-May-2010
CAPWAP(R) 2006-3







Ru =	730.1 kips
Rs =	480.1 kips
Rb =	250.0 kips
Dy =	1.54 in
Dx =	1.59 in



GCC, LOGYARD; Pile: P2 2nd RESTRIKE
PP24x0.401, D62-22; Blow: 54
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 16:33:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 730.1; along Shaft 480.1; at Toe 250.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				730.1				
1	10.0	8.0	2.7	727.4	2.7	0.34	0.05	0.198
2	16.7	14.7	0.4	727.0	3.1	0.06	0.01	0.198
3	23.4	21.4	0.0	727.0	3.1	0.00	0.00	0.000
4	30.1	28.1	0.0	727.0	3.1	0.00	0.00	0.000
5	36.8	34.8	0.0	727.0	3.1	0.00	0.00	0.000
6	43.5	41.5	0.1	726.9	3.2	0.01	0.00	0.198
7	50.2	48.2	0.0	726.9	3.2	0.00	0.00	0.000
8	56.9	54.9	0.0	726.9	3.2	0.00	0.00	0.000
9	63.6	61.6	5.5	721.4	8.7	0.82	0.13	0.198
10	70.3	68.3	16.0	705.4	24.7	2.39	0.38	0.198
11	77.0	75.0	15.4	690.0	40.1	2.30	0.37	0.198
12	83.7	81.7	13.4	676.6	53.5	2.00	0.32	0.198
13	90.4	88.4	19.4	657.2	72.9	2.90	0.46	0.198
14	97.1	95.1	26.3	630.9	99.2	3.93	0.62	0.198
15	103.8	101.8	31.7	599.2	130.9	4.73	0.75	0.198
16	110.5	108.5	41.0	558.2	171.9	6.12	0.97	0.198
17	117.2	115.2	48.9	509.3	220.8	7.30	1.16	0.198
18	123.9	121.9	47.9	461.4	268.7	7.15	1.14	0.198
19	130.6	128.6	49.6	411.8	318.3	7.41	1.18	0.198
20	137.3	135.3	69.0	342.8	387.3	10.30	1.64	0.198
21	144.0	142.0	92.8	250.0	480.1	13.86	2.21	0.198
Avg. Shaft			22.9			3.38	0.54	0.198
Toe			250.0				79.58	0.060

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.125	0.220
Case Damping Factor		1.788	0.284
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	0	

max. Top Comp. Stress = 43.1 ksi (T= 21.3 ms, max= 1.032 x Top)
max. Comp. Stress = 44.4 ksi (Z= 70.3 ft, T= 25.3 ms)
max. Tens. Stress = -1.84 ksi (Z= 70.3 ft, T= 90.9 ms)
max. Energy (EMX) = 91.6 kip-ft; max. Measured Top Displ. (DMX)= 1.59 in

Analysis: 11-May-2010

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	1281.3	-42.3	43.1	-1.42	91.64	23.8	1.618
2	6.7	1283.4	-43.0	43.2	-1.44	91.00	23.7	1.592
5	16.7	1270.2	-45.0	42.7	-1.51	87.72	23.6	1.509
8	26.8	1269.0	-47.0	42.7	-1.58	85.02	23.5	1.418
11	36.8	1272.0	-49.5	42.8	-1.67	82.33	23.3	1.325
14	46.9	1276.0	-51.4	42.9	-1.73	79.55	23.2	1.231
17	56.9	1287.2	-53.1	43.3	-1.78	76.69	22.8	1.135
20	67.0	1302.3	-54.1	43.8	-1.82	72.41	21.9	0.990
23	77.0	1272.9	-53.7	42.8	-1.80	65.37	20.8	0.891
26	87.1	1189.9	-51.3	40.0	-1.73	56.19	19.6	0.794
29	97.1	1188.8	-51.6	40.0	-1.73	50.18	17.9	0.704
32	107.2	1046.8	-43.8	35.2	-1.47	39.52	15.9	0.620
35	117.2	1008.9	-39.6	33.9	-1.33	32.98	13.6	0.544
36	120.6	868.2	-34.5	29.2	-1.16	27.65	12.9	0.520
37	123.9	908.0	-34.6	30.5	-1.16	27.20	12.2	0.498
38	127.3	783.0	-29.6	26.3	-0.99	22.80	11.6	0.478
39	130.6	827.7	-29.5	27.8	-0.99	22.42	10.9	0.458
40	134.0	696.6	-24.5	23.4	-0.82	18.55	10.2	0.440
41	137.3	685.6	-24.5	23.1	-0.82	18.24	10.7	0.422
42	140.7	491.6	-19.4	16.5	-0.65	13.69	10.9	0.406
43	144.0	531.3	-17.3	17.9	-0.58	8.37	10.3	0.391
Absolute	70.3			44.4			(T =	25.3 ms)
	70.3				-1.84		(T =	90.9 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1641.0	1546.6	1452.2	1357.8	1263.5	1169.1	1074.7	980.3	886.0	791.6
RX	1641.0	1546.6	1452.2	1357.8	1263.5	1169.1	1074.7	980.3	886.7	798.8
RU	1631.1	1535.7	1440.3	1345.0	1249.6	1154.2	1058.9	963.5	868.2	772.8

RAU = 290.8 (kips); RA2 = 673.8 (kips)

Current CAPWAP Ru = 730.1 (kips);

Case Method matching requires higher damping factor - please check with PDA-W

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
23.53	21.12	1248.3	1336.3	1341.6	1.593	0.052	0.050	92.4	1349.0

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	29.73	29992.2	492.000	6.283
144.00	29.73	29992.2	492.000	6.283

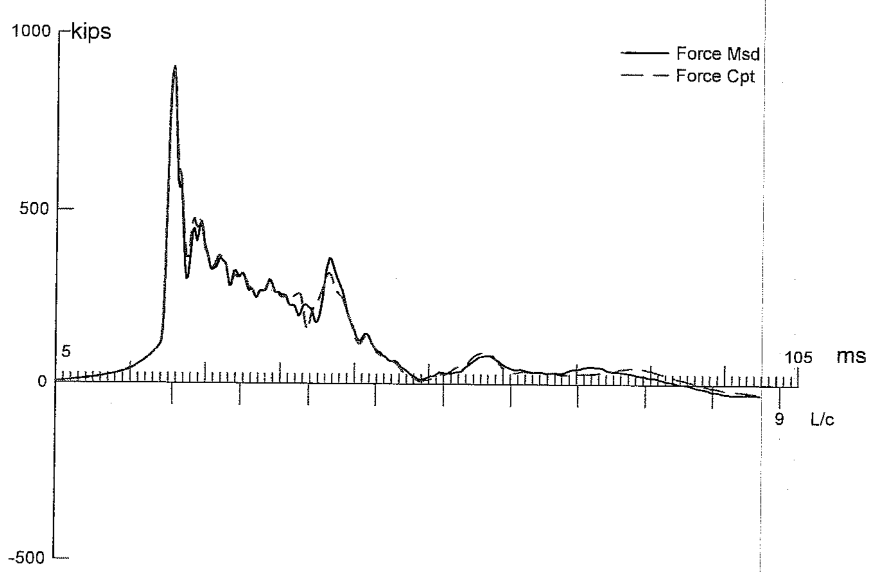
Toe Area 3.142 ft^2

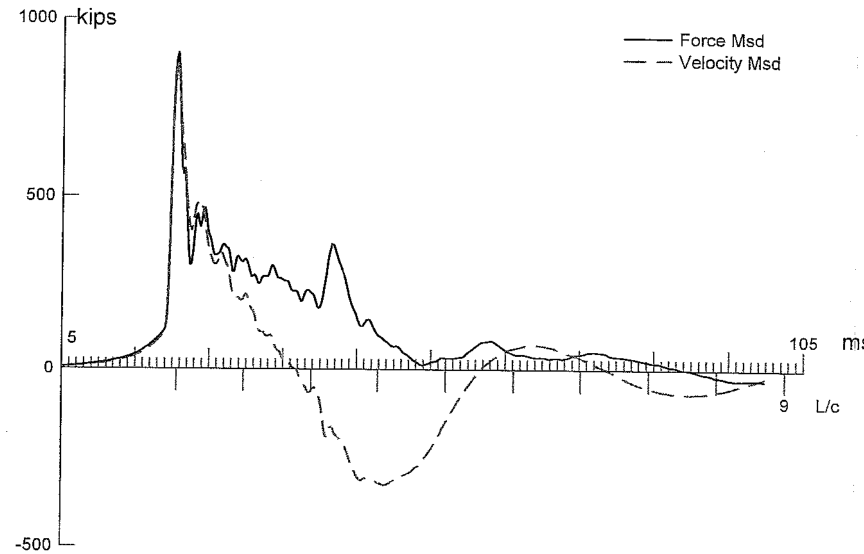
Segmnt Number	Dist. B.G.	Impedance	Imped. Change	Tension Slack	Tension Eff.	Compression Slack	Compression Eff.	Perim.
	ft	kips/ft/s	%	in		in		ft
1	3.35	53.06	0.00	0.000	0.000	0.000	0.000	6.283
8	26.79	53.06	0.00	0.000	0.000	0.000	1.000	6.283
9	30.14	53.06	0.00	0.000	0.000	0.000	0.000	6.283
17	56.93	53.06	0.00	0.000	0.000	-0.050	0.700	6.283
18	60.28	53.06	0.00	0.000	0.000	0.000	0.000	6.283
43	144.00	53.06	0.00	0.000	0.000	0.000	0.000	6.283

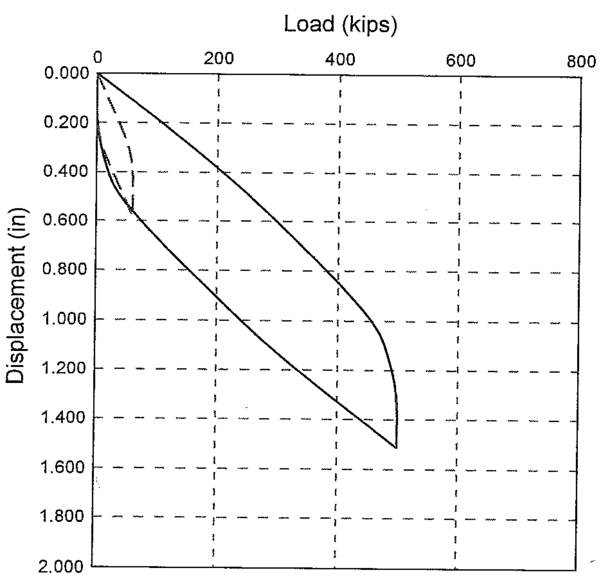
Pile Damping 1.0 %, Time Incr 0.199 ms, Wave Speed 16807.9 ft/s, 2L/c 17.1 ms

GCC, SR520; Pile: Pile 3 End Drive; PP24"x0.401 OPEN END; Blow: 649 (Test: 15-Apr-2010 15:56:)
Robert Miner Dynamic Testing, Inc.

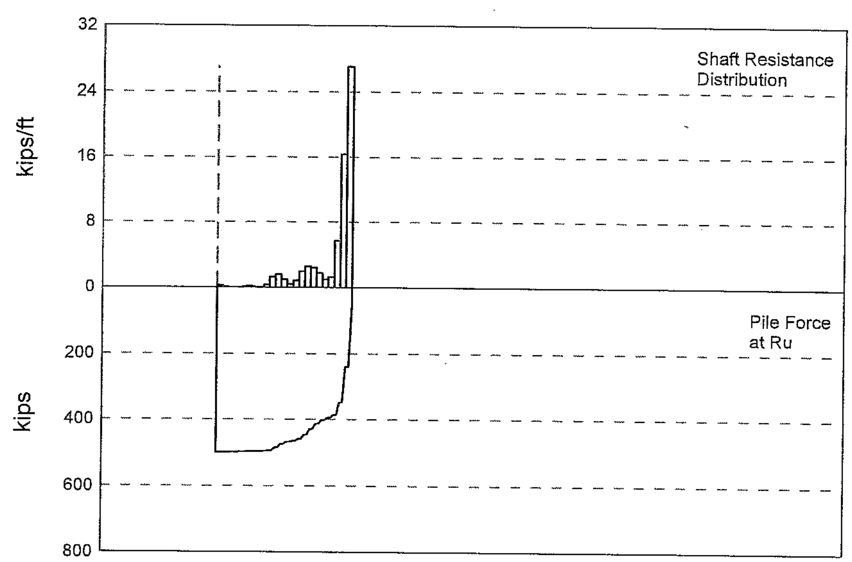
11-May-2010
CAPWAP(R) 2006-3







Ru =	499.9 kips
Rs =	439.9 kips
Rb =	60.0 kips
Dy =	1.36 in
Dx =	1.51 in



GCC, SR520; Pile: Pile 3 End Drive
PP24"x0.401 OPEN END; Blow: 649
Robert Miner Dynamic Testing, Inc.

Test: 15-Apr-2010 15:56:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 499.9; along Shaft 439.9; at Toe 60.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				499.9				
1	6.7	5.7	1.7	498.2	1.7	0.30	0.05	0.094
2	13.3	12.3	0.3	497.9	2.0	0.05	0.01	0.094
3	20.0	19.0	0.0	497.9	2.0	0.00	0.00	0.000
4	26.6	25.6	0.0	497.9	2.0	0.00	0.00	0.000
5	33.3	32.3	0.3	497.6	2.3	0.05	0.01	0.094
6	39.9	38.9	1.1	496.5	3.4	0.17	0.03	0.094
7	46.6	45.6	0.3	496.2	3.7	0.05	0.01	0.094
8	53.2	52.2	0.0	496.2	3.7	0.00	0.00	0.000
9	59.9	58.9	2.1	494.1	5.8	0.32	0.05	0.094
10	66.5	65.5	8.6	485.5	14.4	1.29	0.21	0.094
11	73.2	72.2	10.6	474.9	25.0	1.59	0.25	0.094
12	79.8	78.8	6.5	468.4	31.5	0.98	0.16	0.094
13	86.5	85.5	2.6	465.8	34.1	0.39	0.06	0.094
14	93.1	92.1	5.8	460.0	39.9	0.87	0.14	0.094
15	99.8	98.8	13.1	446.9	53.0	1.97	0.31	0.094
16	106.4	105.4	17.2	429.7	70.2	2.59	0.41	0.094
17	113.1	112.1	16.1	413.6	86.3	2.42	0.39	0.094
18	119.7	118.7	11.8	401.8	98.1	1.77	0.28	0.094
19	126.4	125.4	6.6	395.2	104.7	0.99	0.16	0.094
20	133.0	132.0	8.5	386.7	113.2	1.28	0.20	0.094
21	139.7	138.7	38.0	348.7	151.2	5.71	0.91	0.094
22	146.3	145.3	108.8	239.9	260.0	16.36	2.60	0.094
23	153.0	152.0	179.9	60.0	439.9	27.04	4.30	0.094
Avg. Shaft			19.1			2.89	0.46	0.094
Toe			60.0				290.91	0.165

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.156	0.330
Case Damping Factor		0.778	0.187
Damping Type			Smith
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	16	

max. Top Comp. Stress = 30.2 ksi (T= 21.0 ms, max= 1.020 x Top)
max. Comp. Stress = 30.8 ksi (Z= 66.5 ft, T= 24.7 ms)
max. Tens. Stress = -4.25 ksi (Z= 106.4 ft, T= 60.4 ms)
max. Energy (EMX) = 44.2 kip-ft; max. Measured Top Displ. (DMX)= 1.24 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	897.2	-28.6	30.2	-0.96	44.19	16.6	1.197
2	6.7	897.8	-29.9	30.2	-1.01	44.06	16.6	1.186
5	16.6	894.4	-35.0	30.1	-1.18	43.29	16.5	1.148
8	26.6	896.5	-39.6	30.1	-1.33	42.89	16.5	1.115
11	36.6	899.2	-45.7	30.2	-1.54	42.40	16.4	1.078
14	46.6	899.8	-60.9	30.3	-2.05	41.66	16.3	1.035
17	56.5	905.2	-71.1	30.4	-2.39	40.83	16.1	0.985
20	66.5	914.8	-71.7	30.8	-2.41	39.79	15.8	0.936
23	76.5	881.3	-86.2	29.6	-2.90	36.65	15.5	0.885
26	86.5	875.2	-106.8	29.4	-3.59	34.87	15.3	0.827
29	96.5	873.3	-121.2	29.4	-4.08	32.98	14.9	0.771
32	106.4	868.6	-126.5	29.2	-4.25	30.82	14.4	0.717
35	116.4	812.8	-117.8	27.3	-3.96	26.99	14.0	0.664
38	126.4	802.8	-116.0	27.0	-3.90	25.26	13.6	0.614
39	129.7	795.0	-114.6	26.7	-3.85	24.51	13.5	0.598
40	133.0	808.9	-114.6	27.2	-3.85	24.25	13.2	0.581
41	136.4	808.6	-112.4	27.2	-3.78	23.39	12.8	0.564
42	139.7	841.6	-112.8	28.3	-3.79	23.12	12.2	0.547
43	143.0	772.1	-100.5	26.0	-3.38	20.48	12.1	0.532
44	146.3	718.4	-101.4	24.2	-3.41	20.27	13.4	0.517
45	149.7	441.4	-68.1	14.8	-2.29	13.78	14.1	0.507
46	153.0	490.8	-69.1	16.5	-2.32	3.11	13.6	0.496
Absolute	66.5			30.8			(T =	24.7 ms)
	106.4				-4.25		(T =	60.4 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1044.0	968.0	892.1	816.2	740.2	664.3	588.4	512.4	436.5	360.6
RX	1044.0	968.0	892.1	816.2	740.2	664.3	607.9	588.7	569.4	550.2
RU	973.7	890.7	807.7	724.8	641.8	558.8	475.9	392.9	310.0	227.0

RAU = 382.2 (kips); RA2 = 497.5 (kips)

Current CAPWAP Ru = 499.9 (kips); RMX requires J > 0.9;

Check with PDA-W; RA2 may be a better Case Method

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
16.78	20.78	890.7	912.6	912.6	1.238	0.154	0.154	44.4	766.0

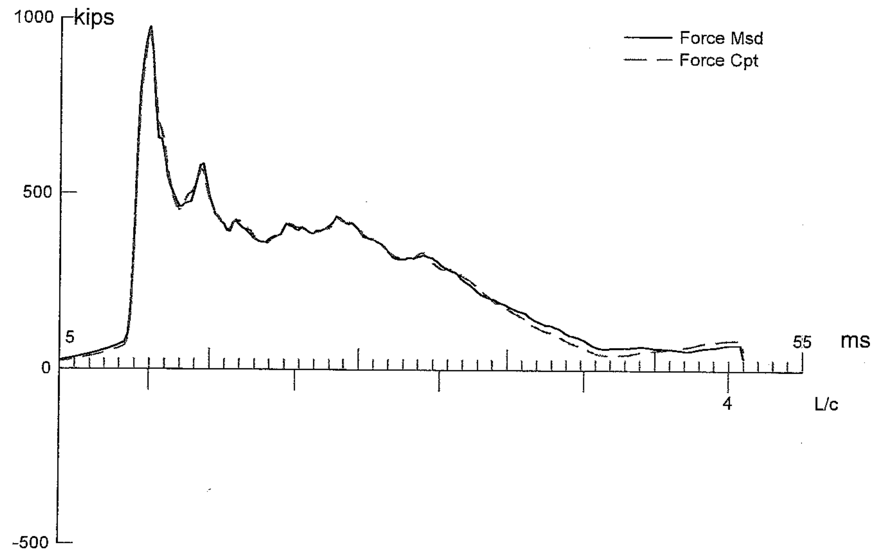
PILE PROFILE AND PILE MODEL

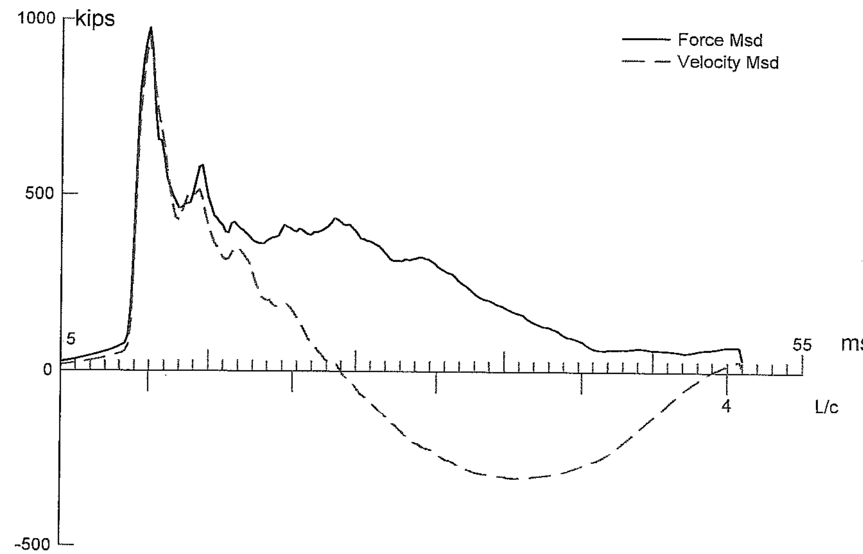
Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	29.73	29992.2	492.000	6.283
153.00	29.73	29992.2	492.000	6.283

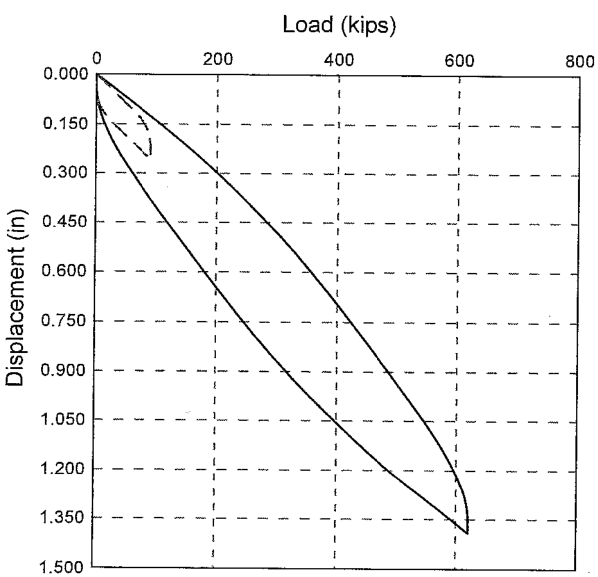
Toe Area 0.206 ft^2

Top Segment Length 3.33 ft, Top Impedance 53.06 kips/ft/s

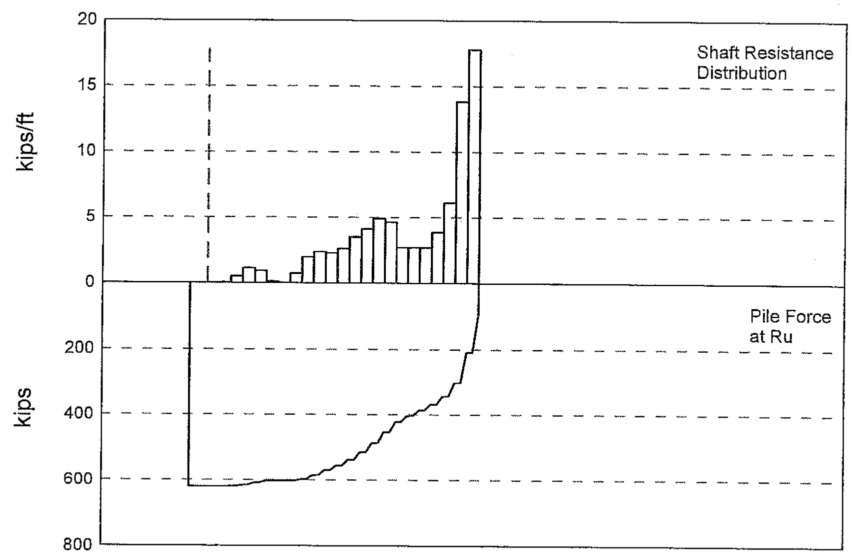
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 16807.9 ft/s, 2L/c 18.2 ms







Ru =	620.4 kips
Rs =	530.4 kips
Rb =	90.0 kips
Dy =	1.34 in
Dx =	1.39 in



CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 620.4; along Shaft 530.4; at Toe 90.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				620.4				
1	16.7	4.7	0.0	620.4	0.0	0.00	0.00	0.000
2	23.4	11.4	0.2	620.2	0.2	0.03	0.00	0.210
3	30.1	18.1	3.4	616.8	3.6	0.51	0.08	0.210
4	36.8	24.8	7.4	609.4	11.0	1.11	0.18	0.210
5	43.5	31.5	6.1	603.3	17.1	0.91	0.15	0.210
6	50.2	38.2	0.6	602.7	17.7	0.09	0.01	0.210
7	56.9	44.9	0.0	602.7	17.7	0.00	0.00	0.000
8	63.6	51.6	4.9	597.8	22.6	0.73	0.12	0.210
9	70.3	58.3	13.1	584.7	35.7	1.96	0.31	0.210
10	77.0	65.0	15.7	569.0	51.4	2.35	0.37	0.210
11	83.7	71.7	15.0	554.0	66.4	2.24	0.36	0.210
12	90.4	78.4	17.4	536.6	83.8	2.60	0.41	0.210
13	97.1	85.1	23.2	513.4	107.0	3.47	0.55	0.210
14	103.8	91.8	27.3	486.1	134.3	4.08	0.65	0.210
15	110.4	98.4	32.8	453.3	167.1	4.90	0.78	0.210
16	117.1	105.1	30.9	422.4	198.0	4.62	0.73	0.210
17	123.8	111.8	18.0	404.4	216.0	2.69	0.43	0.210
18	130.5	118.5	18.0	386.4	234.0	2.69	0.43	0.210
19	137.2	125.2	18.0	368.4	252.0	2.69	0.43	0.210
20	143.9	131.9	25.7	342.7	277.7	3.84	0.61	0.210
21	150.6	138.6	41.1	301.6	318.8	6.14	0.98	0.210
22	157.3	145.3	92.6	209.0	411.4	13.83	2.20	0.210
23	164.0	152.0	119.0	90.0	530.4	17.78	2.83	0.210
Avg. Shaft			23.1			3.49	0.56	0.210
Toe			90.0				436.36	0.090

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.100	0.161
Case Damping Factor		2.099	0.153
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	0	

max. Top Comp. Stress = 32.5 ksi (T= 11.4 ms, max= 1.019 x Top)
max. Comp. Stress = 33.1 ksi (Z= 30.1 ft, T= 12.9 ms)
max. Tens. Stress = -1.61 ksi (Z= 97.1 ft, T= 47.8 ms)
max. Energy (EMX) = 51.0 kip-ft; max. Measured Top Displ. (DMX)= 1.15 in

KIEWIT GENERAL,; Pile: PILE 3 1ST RESTRIKE
PP24x0.50", CLOSED-END, D62-22; Blow: 4
Robert Miner Dynamic Testing, Inc.

Test: 19-Apr-2010 09:49:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	966.4	0.0	32.5	0.00	50.98	18.1	1.135
2	6.7	966.7	0.0	32.5	0.00	50.61	18.0	1.115
5	16.7	968.3	0.0	32.6	0.00	49.45	18.0	1.054
8	26.8	977.1	0.0	32.9	0.00	48.23	17.7	0.992
11	36.8	983.4	-11.7	33.1	-0.39	46.27	17.3	0.931
14	46.9	930.9	-16.3	31.3	-0.55	42.25	17.0	0.869
17	56.9	937.1	-28.7	31.5	-0.96	40.87	16.8	0.805
20	66.9	948.3	-38.5	31.9	-1.30	38.57	16.1	0.736
23	77.0	938.4	-44.0	31.6	-1.48	35.09	15.2	0.667
26	87.0	858.7	-42.6	28.9	-1.43	29.60	14.3	0.599
29	97.1	854.9	-48.0	28.7	-1.61	26.20	13.0	0.531
32	107.1	742.0	-42.1	25.0	-1.41	20.31	11.7	0.470
35	117.1	698.4	-42.9	23.5	-1.44	16.68	10.5	0.411
38	127.2	583.5	-38.0	19.6	-1.28	12.52	9.8	0.354
41	137.2	573.6	-41.0	19.3	-1.38	10.78	8.9	0.304
44	147.3	520.3	-34.5	17.5	-1.16	8.32	7.8	0.257
45	150.6	557.5	-35.7	18.7	-1.20	8.11	7.1	0.242
46	154.0	490.7	-28.5	16.5	-0.96	6.70	6.7	0.230
47	157.3	481.6	-29.3	16.2	-0.98	6.55	6.9	0.217
48	160.7	298.9	-11.8	10.1	-0.40	4.19	7.0	0.208
49	164.0	328.0	-12.0	11.0	-0.40	1.36	6.6	0.198
Absolute	30.1			33.1			(T =	12.9 ms)
	97.1				-1.61		(T =	47.8 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1234.7	1164.0	1093.4	1022.8	952.1	881.5	810.8	740.2	669.6	598.9
RX	1234.7	1164.0	1093.4	1022.8	952.1	881.5	810.8	740.2	670.6	601.2
RU	1255.1	1186.5	1117.9	1049.4	980.8	912.2	843.6	775.0	706.4	637.8

RAU = 48.5 (kips); RA2 = 525.4 (kips)

Current CAPWAP Ru = 620.4 (kips); Corresponding J(RP)= 0.87; J(RX) = 0.87

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
18.05	11.15	957.8	983.3	985.9	1.145	0.050	0.050	51.4	1031.2

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	29.73	29992.2	492.000	6.283
164.00	29.73	29992.2	492.000	6.283

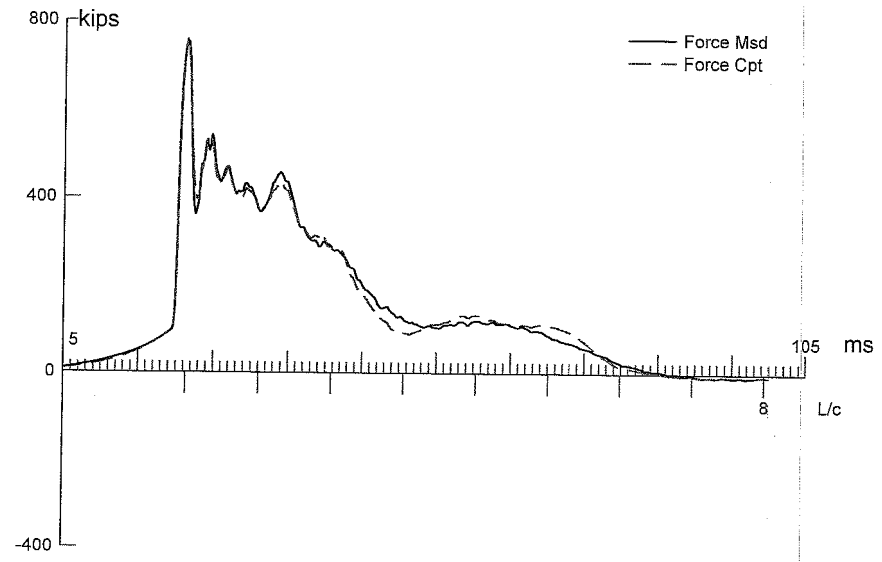
Toe Area 0.206 ft^2

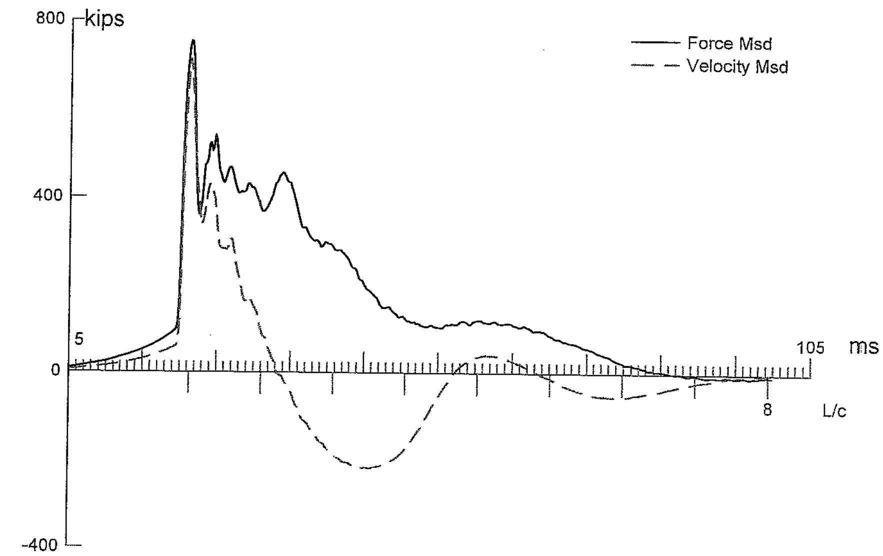
Top Segment Length 3.35 ft, Top Impedance 53.06 kips/ft/s

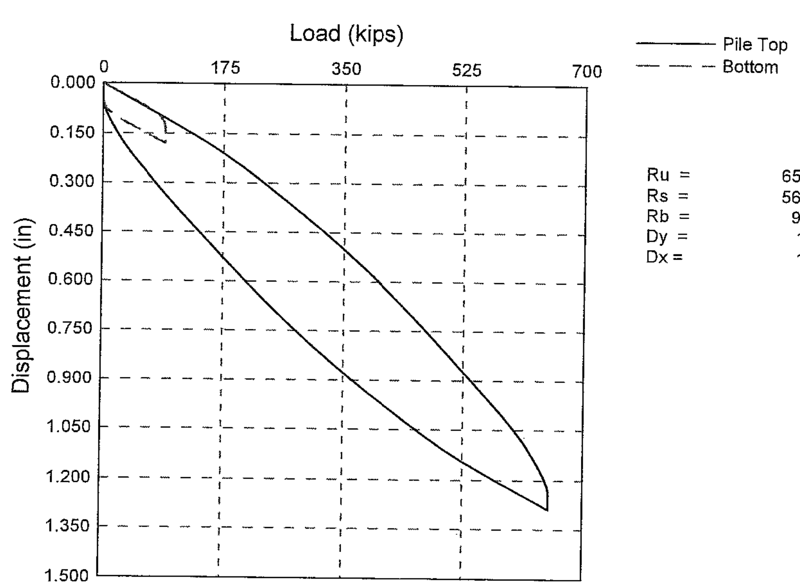
Pile Damping 1.0 %, Time Incr 0.199 ms, Wave Speed 16807.9 ft/s, 2L/c 19.5 ms

GCC; Pile: P3 2nd RESTRIKE; PP24x0.401, D62-22; Blow: 1 (Test: 26-Apr-2010 15:20:)
Robert Miner Dynamic Testing, Inc.

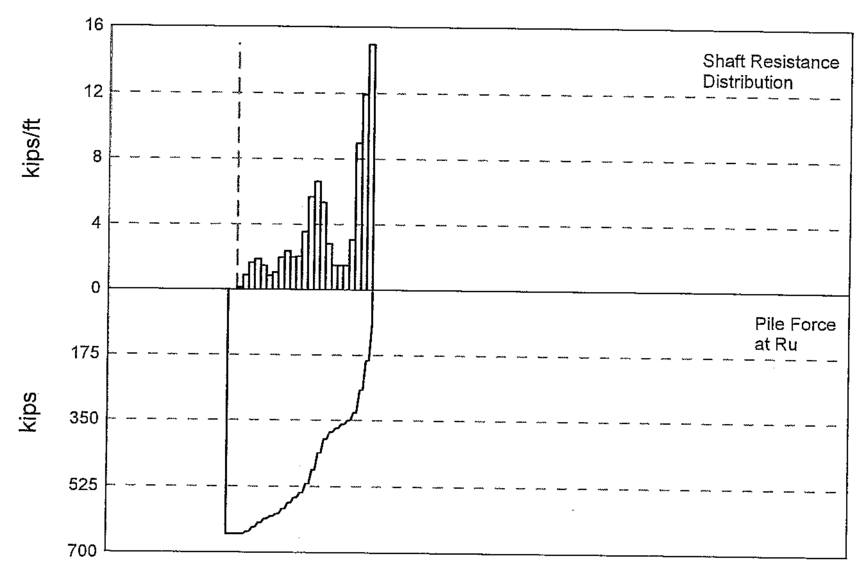
11-May-2010
CAPWAP(R) 2006-3







Ru = 650.6 kips
Rs = 560.6 kips
Rb = 90.0 kips
Dy = 1.24 in
Dx = 1.29 in



GCC; Pile: P3 2nd RESTRIKE
PP24x0.401, D62-22; Blow: 1
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 15:20:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 650.6; along Shaft 560.6; at Toe 90.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft	Quake in
				650.6					
1	16.7	4.7	0.8	649.8	0.8	0.17	0.03	0.170	0.100
2	23.4	11.4	5.8	644.0	6.6	0.87	0.14	0.170	0.100
3	30.1	18.1	11.0	633.0	17.6	1.64	0.26	0.170	0.100
4	36.8	24.8	12.6	620.4	30.2	1.88	0.30	0.170	0.100
5	43.5	31.5	9.8	610.6	40.0	1.46	0.23	0.170	0.100
6	50.2	38.2	5.7	604.9	45.7	0.85	0.14	0.170	0.100
7	56.9	44.9	6.9	598.0	52.6	1.03	0.16	0.170	0.100
8	63.6	51.6	13.2	584.8	65.8	1.97	0.31	0.170	0.100
9	70.3	58.3	16.0	568.8	81.8	2.39	0.38	0.170	0.100
10	77.0	65.0	13.3	555.5	95.1	1.99	0.32	0.170	0.100
11	83.7	71.7	13.7	541.8	108.8	2.05	0.33	0.170	0.100
12	90.4	78.4	23.8	518.0	132.6	3.56	0.57	0.170	0.100
13	97.1	85.1	38.2	479.8	170.8	5.71	0.91	0.170	0.100
14	103.8	91.8	44.4	435.4	215.2	6.63	1.06	0.170	0.100
15	110.4	98.4	35.9	399.5	251.1	5.36	0.85	0.170	0.100
16	117.1	105.1	18.9	380.6	270.0	2.82	0.45	0.170	0.100
17	123.8	111.8	10.0	370.6	280.0	1.49	0.24	0.170	0.100
18	130.5	118.5	10.0	360.6	290.0	1.49	0.24	0.170	0.100
19	137.2	125.2	10.0	350.6	300.0	1.49	0.24	0.170	0.100
20	143.9	131.9	20.6	330.0	320.6	3.08	0.49	0.170	0.089
21	150.6	138.6	60.0	270.0	380.6	8.96	1.43	0.170	0.074
22	157.3	145.3	80.0	190.0	460.6	11.95	1.90	0.170	0.062
23	164.0	152.0	100.0	90.0	560.6	14.94	2.38	0.170	0.051
Avg. Shaft			24.4			3.69	0.59	0.170	0.083
Toe			90.0				435.92	0.130	0.100

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.796	0.221
Unloading Quake	(% of loading quake)	50	100
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	11	

max. Top Comp. Stress = 25.6 ksi (T= 21.7 ms, max= 1.029 x Top)
max. Comp. Stress = 26.3 ksi (Z= 23.4 ft, T= 22.9 ms)
max. Tens. Stress = -0.34 ksi (Z= 30.1 ft, T= 93.0 ms)
max. Energy (EMX) = 36.1 kip-ft; max. Measured Top Displ. (DMX)= 0.93 in

Analysis: 11-May-2010

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	760.3	-8.0	25.6	-0.27	36.13	13.3	0.931
2	6.7	762.0	-8.4	25.6	-0.28	35.78	13.3	0.912
5	16.7	771.9	-9.3	26.0	-0.31	34.71	13.1	0.853
8	26.8	771.1	-9.4	25.9	-0.32	32.74	12.7	0.794
11	36.8	760.2	-9.4	25.6	-0.32	30.27	12.2	0.736
14	46.9	707.2	-7.1	23.8	-0.24	26.67	11.9	0.679
17	56.9	710.0	-7.2	23.9	-0.24	25.14	11.5	0.625
20	66.9	680.2	-5.7	22.9	-0.19	22.32	10.9	0.569
23	77.0	661.8	-4.5	22.3	-0.15	19.92	10.4	0.509
26	87.0	631.7	-1.1	21.2	-0.04	17.03	9.7	0.452
29	97.1	630.9	0.0	21.2	0.00	14.60	8.5	0.395
32	107.1	493.2	0.0	16.6	0.00	9.86	7.5	0.343
35	117.1	442.6	0.0	14.9	0.00	7.76	7.0	0.296
38	127.2	399.3	0.0	13.4	0.00	6.06	6.7	0.247
41	137.2	399.8	0.0	13.4	0.00	5.17	6.3	0.202
44	147.3	403.2	0.0	13.6	0.00	4.00	5.5	0.159
45	150.6	424.7	0.0	14.3	0.00	3.81	5.0	0.144
46	154.0	335.7	0.0	11.3	0.00	2.85	4.8	0.132
47	157.3	327.9	0.0	11.0	0.00	2.73	5.0	0.121
48	160.7	228.4	0.0	7.7	0.00	1.76	4.9	0.113
49	164.0	259.9	0.0	8.7	0.00	0.72	4.6	0.104
Absolute	23.4			26.3			(T =	22.9 ms)
	30.1				-0.34		(T =	93.0 ms)

GCC; Pile: P3 2nd RESTRIKE
PP24x0.401, D62-22; Blow: 1
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 15:20:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	966.4	916.5	866.5	816.6	766.7	716.7	666.8	616.9	567.0	517.0
RX	967.8	917.5	867.2	816.9	766.6	716.3	666.0	615.9	566.0	516.1
RU	1079.9	1040.8	1001.7	962.6	923.5	884.4	845.3	806.2	767.2	728.1

RAU = 71.8 (kips); RA2 = 593.6 (kips)

Current CAPWAP Ru = 650.6 (kips); Corresponding J(RP)= 0.63; J(RX) = 0.63

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
13.57	21.51	714.1	751.6	758.1	0.932	0.053	0.050	36.2	884.1

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	29.73	29992.2	492.000	6.283
164.00	29.73	29992.2	492.000	6.283

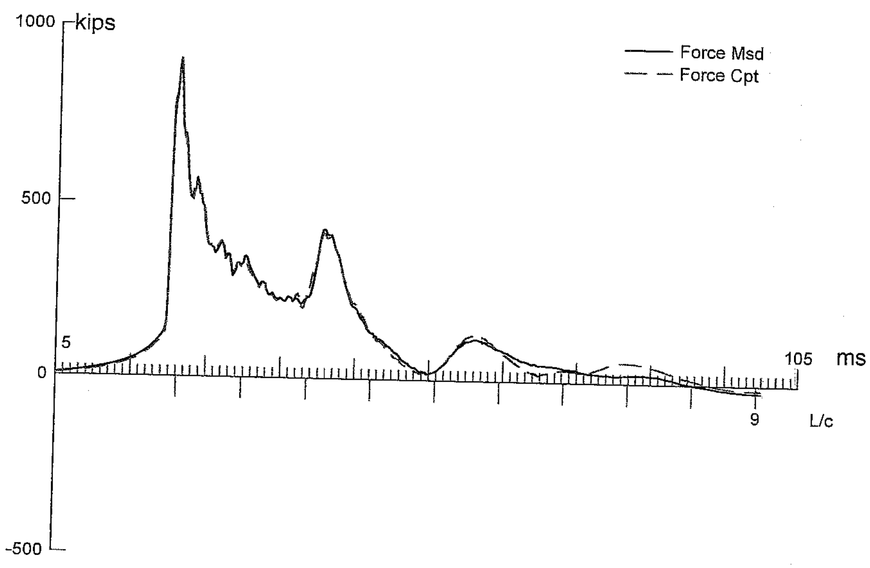
Toe Area 0.206 ft^2

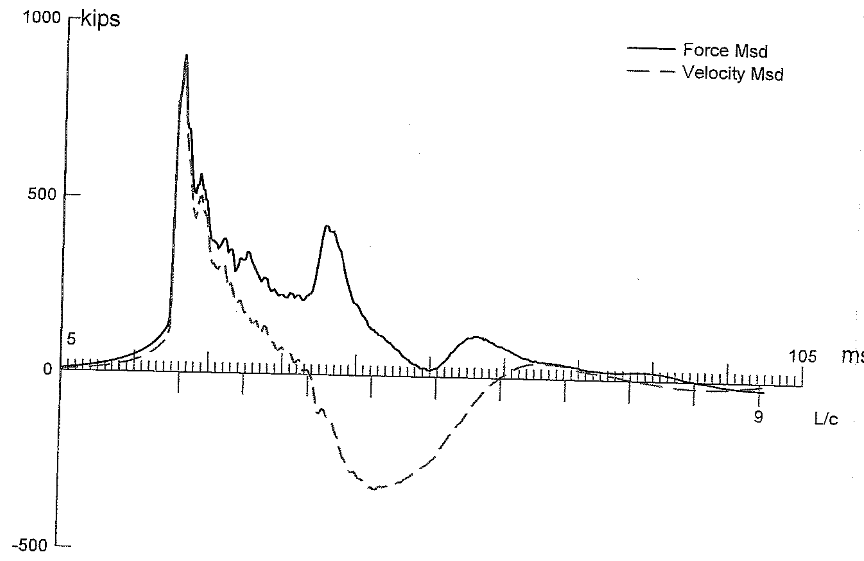
Top Segment Length 3.35 ft, Top Impedance 53.06 kips/ft/s

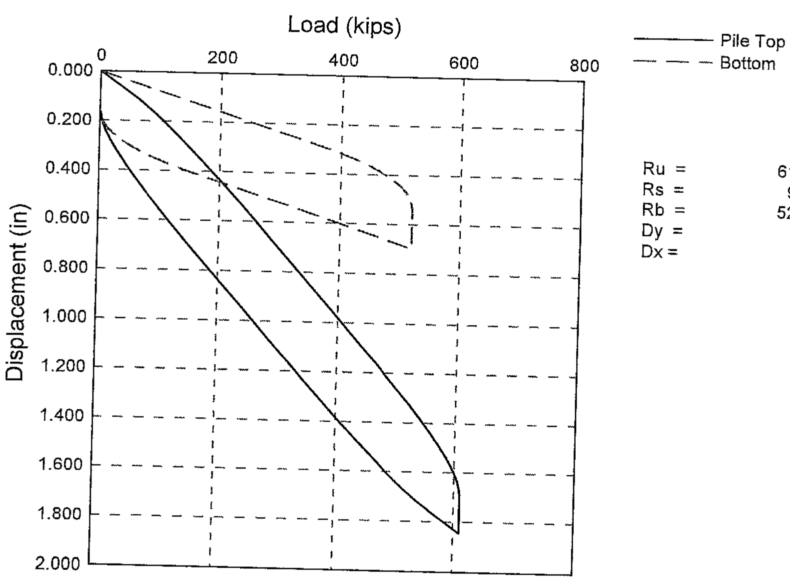
Pile Damping 1.0 %, Time Incr 0.199 ms, Wave Speed 16807.9 ft/s, 2L/c 19.5 ms

KIEWIT GENERAL; Pile: PILE 4 End Drive; PP24x0.401", D46-32; Blow: 2086 (Test: 22-Apr-2010 10:00:)
Robert Miner Dynamic Testing, Inc.

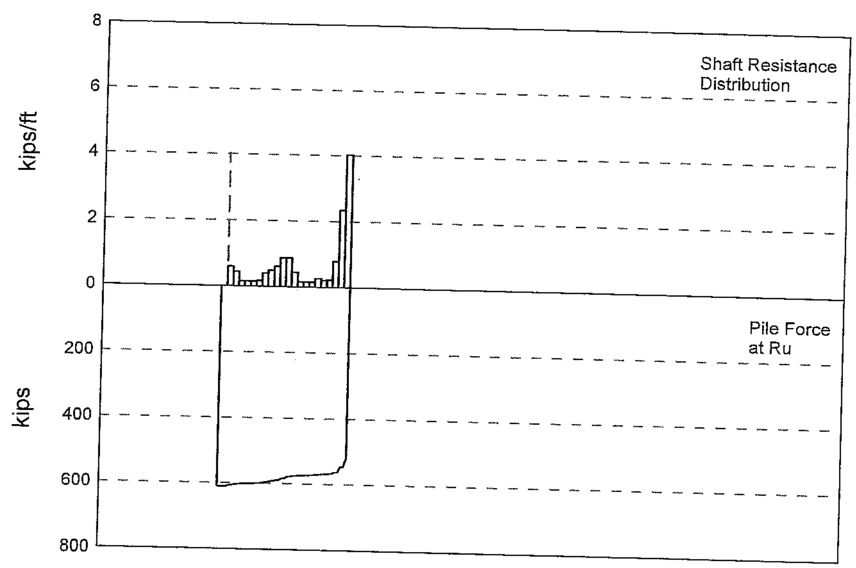
11-May-2010
CAPWAP(R) 2006-3







Ru =	610.2 kips
Rs =	90.2 kips
Rb =	520.0 kips
Dy =	1.68 in
Dx =	1.85 in



KIEWIT GENERAL; Pile: PILE 4 End Drive
PP24x0.401", D46-32; Blow: 2086
Robert Miner Dynamic Testing, Inc.

Test: 22-Apr-2010 10:00:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 610.2; along Shaft 90.2; at Toe 520.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				610.2				
1	13.3	6.3	3.9	606.3	3.9	0.62	0.10	0.250
2	19.9	12.9	2.9	603.4	6.8	0.44	0.07	0.250
3	26.5	19.5	1.0	602.4	7.8	0.15	0.02	0.250
4	33.2	26.2	1.0	601.4	8.8	0.15	0.02	0.250
5	39.8	32.8	1.0	600.4	9.8	0.15	0.02	0.250
6	46.5	39.5	1.1	599.3	10.9	0.17	0.03	0.250
7	53.1	46.1	2.6	596.7	13.5	0.39	0.06	0.250
8	59.7	52.7	3.2	593.5	16.7	0.48	0.08	0.250
9	66.4	59.4	4.0	589.5	20.7	0.60	0.10	0.250
10	73.0	66.0	5.8	583.7	26.5	0.87	0.14	0.250
11	79.6	72.6	5.8	577.9	32.3	0.87	0.14	0.250
12	86.3	79.3	2.9	575.0	35.2	0.44	0.07	0.250
13	92.9	85.9	1.0	574.0	36.2	0.15	0.02	0.250
14	99.5	92.5	1.0	573.0	37.2	0.15	0.02	0.250
15	106.2	99.2	1.0	572.0	38.2	0.15	0.02	0.250
16	112.8	105.8	1.7	570.3	39.9	0.26	0.04	0.250
17	119.5	112.5	1.4	568.9	41.3	0.21	0.03	0.250
18	126.1	119.1	1.5	567.4	42.8	0.23	0.04	0.250
19	132.7	125.7	5.2	562.2	48.0	0.78	0.12	0.250
20	139.4	132.4	15.5	546.7	63.5	2.34	0.37	0.250
21	146.0	139.0	26.7	520.0	90.2	4.02	0.64	0.250
Avg. Shaft			4.3			0.65	0.10	0.250
Toe			520.0				165.52	0.070

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.100	0.410
Case Damping Factor		0.425	0.686
Unloading Quake	(% of loading quake)	40	70
Reloading Level	(% of Ru)	100	100

max. Top Comp. Stress = 30.5 ksi (T= 21.5 ms, max= 1.013 x Top)
max. Comp. Stress = 30.9 ksi (Z= 13.3 ft, T= 22.1 ms)
max. Tens. Stress = -3.38 ksi (Z= 132.7 ft, T= 62.0 ms)
max. Energy (EMX) = 48.8 kip-ft; max. Measured Top Displ. (DMX)= 1.23 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	908.4	-13.1	30.5	-0.44	48.75	16.6	1.239
2	6.6	913.3	-13.6	30.7	-0.46	48.65	16.5	1.229
5	16.6	901.7	-10.7	30.3	-0.36	47.14	16.3	1.194
8	26.5	892.9	-8.8	30.0	-0.29	45.88	16.2	1.157
11	36.5	887.9	-7.4	29.9	-0.25	44.85	16.0	1.118
14	46.5	891.4	-7.4	30.0	-0.25	44.04	15.8	1.077
17	56.4	883.8	-4.3	29.7	-0.14	42.39	15.6	1.029
20	66.4	885.2	-17.9	29.8	-0.60	40.79	15.3	0.978
23	76.3	852.8	-29.1	28.7	-0.98	37.57	15.0	0.922
26	86.3	834.4	-45.4	28.1	-1.53	35.13	14.8	0.860
29	96.2	821.3	-60.9	27.6	-2.05	33.13	14.6	0.798
32	106.2	824.3	-78.4	27.7	-2.64	31.63	14.5	0.733
35	116.1	820.2	-90.7	27.6	-3.05	29.63	14.3	0.663
36	119.5	823.9	-95.3	27.7	-3.20	29.09	14.2	0.639
37	122.8	821.5	-97.9	27.6	-3.29	28.31	14.1	0.614
38	126.1	829.1	-99.2	27.9	-3.34	27.73	14.0	0.589
39	129.4	830.5	-97.4	27.9	-3.27	26.92	13.8	0.564
40	132.7	847.4	-100.6	28.5	-3.38	26.31	13.5	0.539
41	136.0	823.6	-94.1	27.7	-3.16	25.05	13.5	0.513
42	139.4	804.8	-94.2	27.1	-3.17	24.45	13.8	0.488
43	142.7	761.5	-74.9	25.6	-2.52	22.14	13.3	0.463
44	146.0	793.1	-74.7	26.7	-2.51	19.75	12.7	0.439
Absolute	13.3			30.9			(T =	22.1 ms)
	132.7				-3.38		(T =	62.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	924.2	852.2	780.1	708.1	636.1	564.1	492.0	420.0	348.0	276.0
RX	1020.1	940.3	860.4	780.6	700.7	672.2	644.8	617.4	590.0	562.6
RU	924.2	852.2	780.1	708.1	636.1	564.1	492.0	420.0	348.0	276.0

RAU = 408.0 (kips); RA2 = 581.4 (kips)

Current CAPWAP Ru = 610.2 (kips); Corresponding J(RP)= 0.44; J(RX) = 0.73

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
17.11	21.32	814.8	829.6	910.7	1.230	0.155	0.162	48.9	843.5

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	29.73	29992.2	492.000	6.283
146.00	29.73	29992.2	492.000	6.283

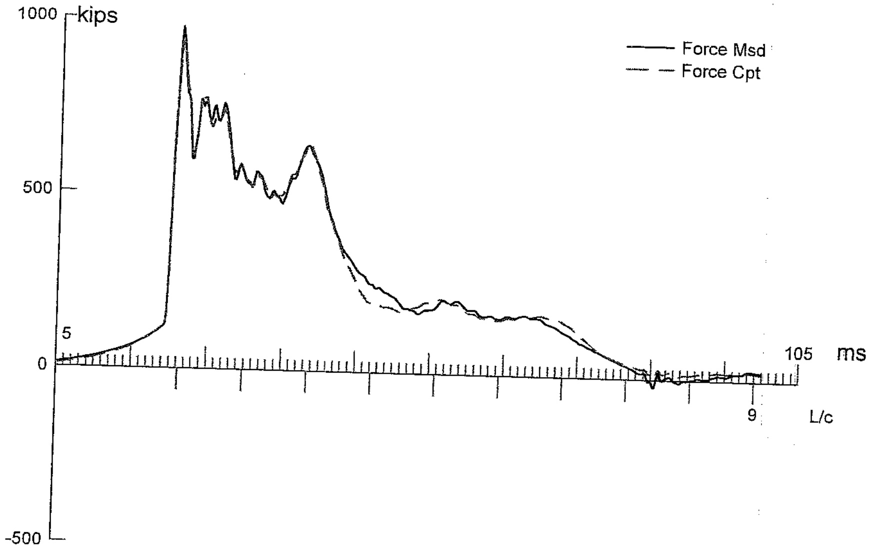
Toe Area 3.142 ft^2

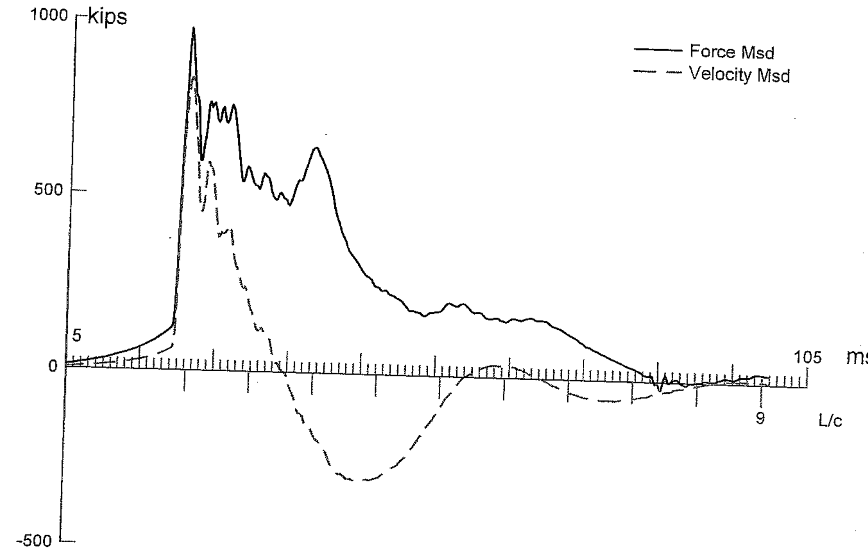
Top Segment Length 3.32 ft, Top Impedance 53.06 kips/ft/s

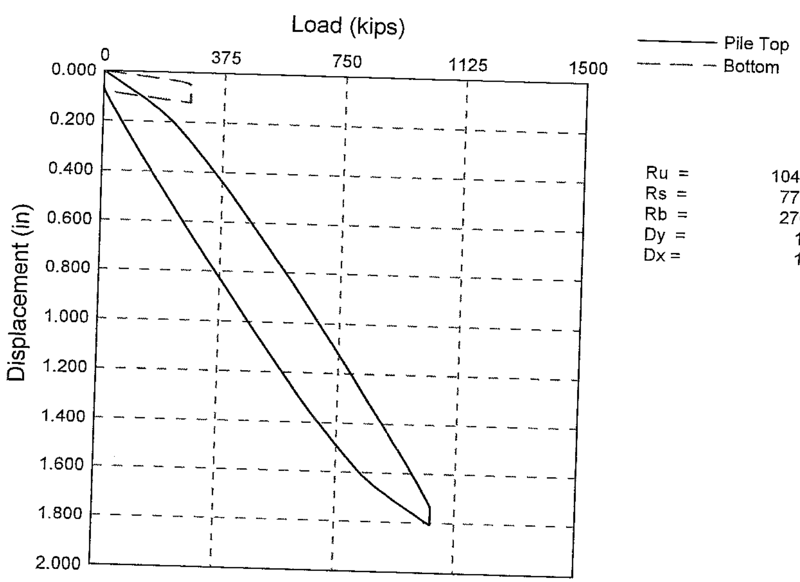
Pile Damping 1.0 %, Time Incr 0.197 ms, Wave Speed 16807.9 ft/s, 2L/c 17.4 ms

GCC,; Pile: P4 1ST RESTRIKE; PP24x0.401, D62-22; Blow: 7 (Test: 26-Apr-2010 09:53:)
Robert Miner Dynamic Testing, Inc.

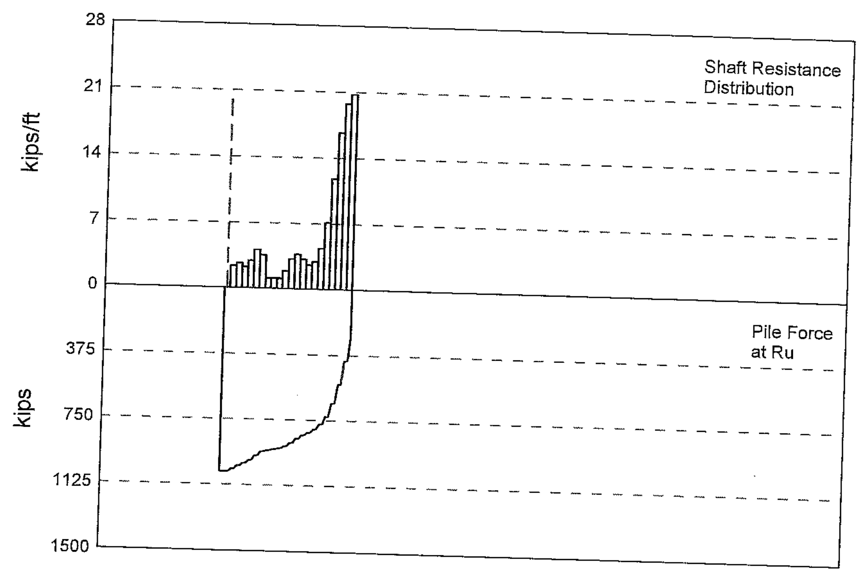
11-May-2010
CAPWAP(R) 2006-3







Ru = 1049.9 kips
Rs = 779.9 kips
Rb = 270.0 kips
Dy = 1.74 in
Dx = 1.81 in



GCC,; Pile: P4 1ST RESTRIKE
PP24x0.401, D62-22; Blow: 7
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 09:53:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1049.9; along Shaft 779.9; at Toe 270.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft	Quake in
				1049.9					
1	13.2	7.0	15.0	1034.9	15.0	2.14	0.34	0.130	0.100
2	19.8	13.6	17.1	1017.8	32.1	2.59	0.41	0.130	0.100
3	26.4	20.2	14.4	1003.4	46.5	2.18	0.35	0.130	0.100
4	33.0	26.8	18.8	984.6	65.3	2.85	0.45	0.130	0.100
5	39.6	33.4	26.9	957.7	92.2	4.08	0.65	0.130	0.100
6	46.2	40.0	23.1	934.6	115.3	3.50	0.56	0.130	0.100
7	52.8	46.6	7.0	927.6	122.3	1.06	0.17	0.130	0.100
8	59.4	53.2	7.0	920.6	129.3	1.06	0.17	0.130	0.100
9	66.0	59.8	7.0	913.6	136.3	1.06	0.17	0.130	0.100
10	72.6	66.4	12.0	901.6	148.3	1.82	0.29	0.130	0.100
11	79.2	73.0	20.5	881.1	168.8	3.11	0.49	0.130	0.100
12	85.8	79.6	23.8	857.3	192.6	3.61	0.57	0.130	0.100
13	92.4	86.2	20.5	836.8	213.1	3.11	0.49	0.130	0.100
14	99.0	92.8	16.5	820.3	229.6	2.50	0.40	0.130	0.100
15	105.6	99.4	18.8	801.5	248.4	2.85	0.45	0.130	0.100
16	112.2	106.0	28.5	773.0	276.9	4.32	0.69	0.130	0.090
17	118.8	112.6	47.2	725.8	324.1	7.15	1.14	0.130	0.080
18	125.4	119.2	77.7	648.1	401.8	11.77	1.87	0.130	0.070
19	132.0	125.8	110.4	537.7	512.2	16.73	2.66	0.130	0.060
20	138.6	132.4	130.6	407.1	642.8	19.79	3.15	0.130	0.050
21	145.2	139.0	137.1	270.0	779.9	20.77	3.31	0.130	0.029
Avg. Shaft			37.1			5.61	0.89	0.130	0.069
Toe			270.0				85.94	0.150	0.040

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		1.911	0.763
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	10	

max. Top Comp. Stress = 31.8 ksi (T= 21.4 ms, max= 1.031 x Top)
max. Comp. Stress = 32.8 ksi (Z= 13.2 ft, T= 22.0 ms)
max. Tens. Stress = 0.00 ksi (Z= 3.3 ft, T= 0.0 ms)
max. Energy (EMX) = 67.9 kip-ft; max. Measured Top Displ. (DMX)= 1.23 in

GCC,; Pile: P4 1ST RESTRIKE
PP24x0.401, D62-22; Blow: 7
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 09:53:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	945.1	0.0	31.8	0.00	67.92	16.1	1.218
2	6.6	954.2	0.0	32.1	0.00	67.43	15.9	1.195
5	16.5	937.3	0.0	31.5	0.00	62.96	15.3	1.124
8	26.4	918.0	0.0	30.9	0.00	58.14	14.7	1.051
11	36.3	864.2	0.0	29.1	0.00	50.67	13.9	0.973
14	46.2	819.7	0.0	27.6	0.00	44.74	13.3	0.897
17	56.1	752.3	0.0	25.3	0.00	38.73	13.0	0.822
20	66.0	752.4	0.0	25.3	0.00	35.75	12.6	0.740
23	75.9	738.5	0.0	24.8	0.00	31.54	12.0	0.657
26	85.8	730.8	0.0	24.6	0.00	27.13	11.2	0.567
29	95.7	770.3	0.0	25.9	0.00	20.68	10.6	0.468
32	105.6	769.4	0.0	25.9	0.00	16.45	9.9	0.366
35	115.5	726.6	0.0	24.4	0.00	11.59	8.8	0.271
36	118.8	750.0	0.0	25.2	0.00	10.67	8.1	0.240
37	122.1	709.5	0.0	23.9	0.00	8.48	7.6	0.209
38	125.4	709.9	0.0	23.9	0.00	7.64	6.8	0.179
39	128.7	632.4	0.0	21.3	0.00	5.51	6.2	0.153
40	132.0	633.2	0.0	21.3	0.00	4.89	5.4	0.127
41	135.3	532.5	0.0	17.9	0.00	3.15	4.8	0.106
42	138.6	523.6	0.0	17.6	0.00	2.69	4.2	0.083
43	141.9	423.4	0.0	14.2	0.00	1.55	3.7	0.065
44	145.2	442.5	0.0	14.9	0.00	0.91	2.7	0.047
Absolute	13.2			32.8			(T =	22.0 ms)
	3.3				0.00		(T =	0.0 ms)

Analysis: 11-May-2010

GCC,; Pile: P4 1ST RESTRIKE
PP24x0.401, D62-22; Blow: 7
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 09:53:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1304.9	1254.0	1203.1	1152.2	1101.2	1050.3	999.4	948.5	897.5	846.6
RX	1304.9	1254.0	1203.1	1152.2	1101.2	1050.3	999.4	948.5	897.5	846.6
RU	1367.5	1322.8	1278.2	1233.5	1188.9	1144.2	1099.5	1054.9	1010.2	965.5

RAU = 221.4 (kips); RA2 = 916.1 (kips)

Current CAPWAP Ru = 1049.9 (kips); Corresponding J(RP)= 0.50; J(RX) = 0.50

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips
15.81	21.40	838.9	975.2	987.0	1.227	0.069	0.071	68.3	1262.6

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	29.73	29992.2	492.000	6.283
145.20	29.73	29992.2	492.000	6.283

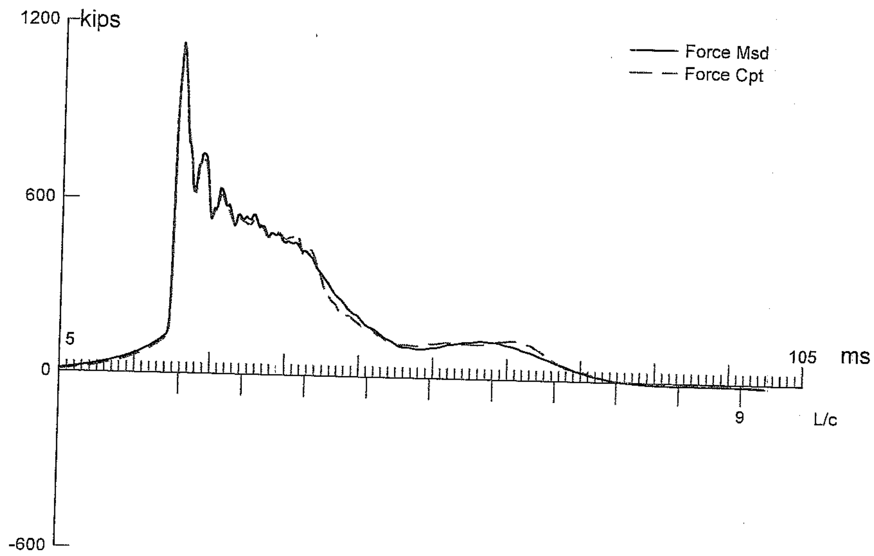
Toe Area 3.142 ft^2

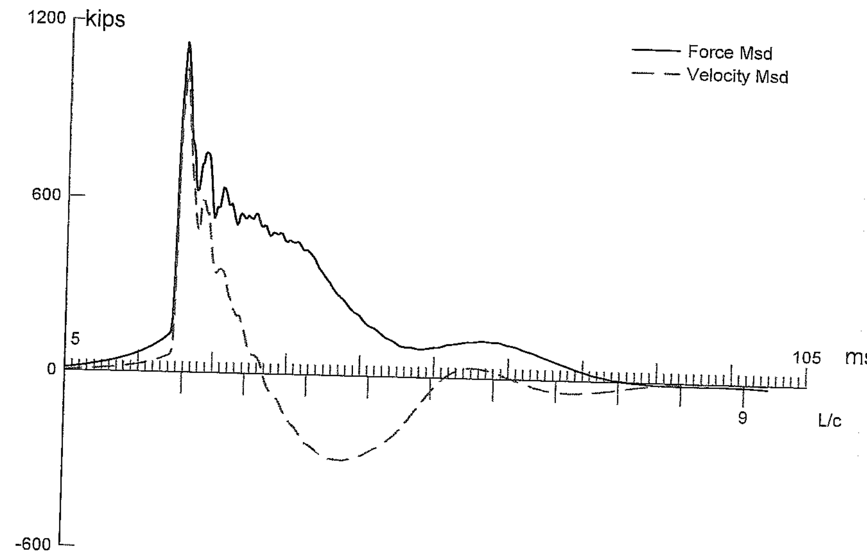
Top Segment Length 3.30 ft, Top Impedance 53.06 kips/ft/s

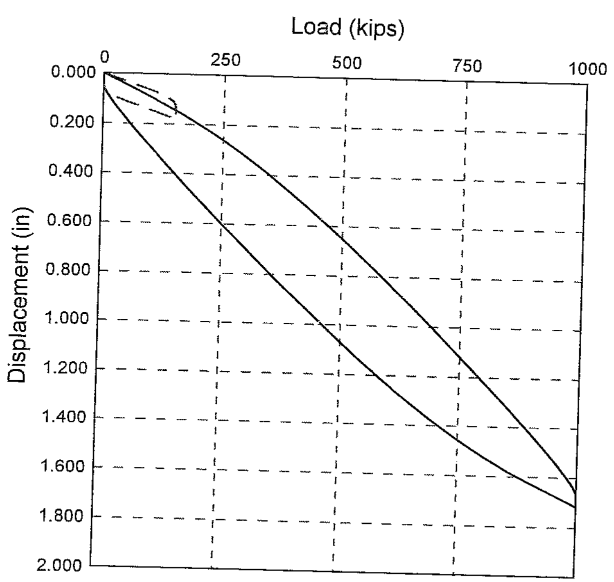
Pile Damping 1.0 %, Time Incr 0.196 ms, Wave Speed 16807.9 ft/s, 2L/c 17.3 ms

GCC; Pile: P4 2nd Restrike; PP24x0.401, D62-22; Blow: 4 (Test: 03-May-2010 10:53:)
Robert Miner Dynamic Testing, Inc.

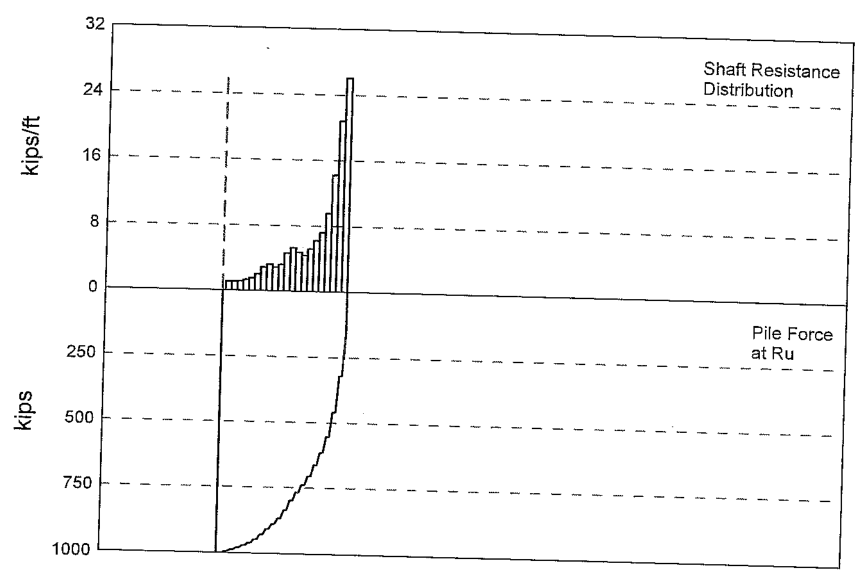
11-May-2010
CAPWAP(R) 2006-3







Ru = 999.9 kips
Rs = 849.9 kips
Rb = 150.0 kips
Dy = 1.67 in
Dx = 1.72 in



GCC; Pile: P4 2nd Restrike
PP24x0.401, D62-22; Blow: 4
Robert Miner Dynamic Testing, Inc.

Test: 03-May-2010 10:53:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 999.9; along Shaft 849.9; at Toe 150.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				999.9				
1	9.9	7.4	7.1	992.8	7.1	0.96	0.15	0.120
2	16.5	14.0	7.1	985.7	14.2	1.08	0.17	0.120
3	23.0	20.5	7.5	978.2	21.7	1.14	0.18	0.120
4	29.6	27.1	8.7	969.5	30.4	1.32	0.21	0.120
5	36.2	33.7	10.2	959.3	40.6	1.55	0.25	0.120
6	42.8	40.3	13.4	945.9	54.0	2.04	0.32	0.120
7	49.4	46.9	19.1	926.8	73.1	2.90	0.46	0.120
8	55.9	53.4	21.5	905.3	94.6	3.27	0.52	0.120
9	62.5	60.0	19.1	886.2	113.7	2.90	0.46	0.120
10	69.1	66.6	21.4	864.8	135.1	3.25	0.52	0.120
11	75.7	73.2	30.6	834.2	165.7	4.65	0.74	0.120
12	82.3	79.8	35.2	799.0	200.9	5.35	0.85	0.120
13	88.8	86.3	31.3	767.7	232.2	4.76	0.76	0.120
14	95.4	92.9	29.0	738.7	261.2	4.41	0.70	0.120
15	102.0	99.5	34.4	704.3	295.6	5.23	0.83	0.120
16	108.6	106.1	41.1	663.2	336.7	6.24	0.99	0.120
17	115.2	112.7	47.6	615.6	384.3	7.23	1.15	0.120
18	121.8	119.3	62.8	552.8	447.1	9.54	1.52	0.120
19	128.3	125.8	93.8	459.0	540.9	14.25	2.27	0.120
20	134.9	132.4	137.1	321.9	678.0	20.83	3.32	0.120
21	141.5	139.0	171.9	150.0	849.9	26.12	4.16	0.120
Avg. Shaft			40.5			6.11	0.97	0.120
Toe			150.0				47.75	0.050

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.100	0.100
Case Damping Factor		1.922	0.141
Unloading Quake	(% of loading quake)	30	100
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	6	

max. Top Comp. Stress = 37.7 ksi (T= 21.1 ms, max= 1.010 x Top)
max. Comp. Stress = 38.1 ksi (Z= 9.9 ft, T= 21.5 ms)
max. Tens. Stress = -0.29 ksi (Z= 3.3 ft, T= 100.0 ms)
max. Energy (EMX) = 67.6 kip-ft; max. Measured Top Displ. (DMX)= 1.15 in

Analysis: 11-May-2010

GCC; Pile: P4 2nd Restrike
PP24x0.401, D62-22; Blow: 4
Robert Miner Dynamic Testing, Inc.

Test: 03-May-2010 10:53:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	1122.5	-8.5	37.7	-0.29	67.63	19.6	1.171
2	6.6	1127.3	-8.5	37.9	-0.29	67.13	19.5	1.148
5	16.5	1120.6	-7.5	37.7	-0.25	64.08	19.1	1.076
8	26.3	1092.5	-6.6	36.7	-0.22	59.60	18.7	1.002
11	36.2	1093.1	-6.1	36.8	-0.20	56.23	18.2	0.924
14	46.1	1055.4	-3.7	35.5	-0.12	50.68	17.4	0.846
17	55.9	1040.8	-2.2	35.0	-0.08	45.98	16.6	0.765
20	65.8	963.8	0.0	32.4	0.00	38.95	15.8	0.683
23	75.7	963.0	0.0	32.4	0.00	34.55	14.7	0.600
26	85.6	843.0	0.0	28.3	0.00	26.90	13.6	0.522
29	95.4	818.7	0.0	27.5	0.00	22.61	12.5	0.442
32	105.3	725.2	0.0	24.4	0.00	16.93	11.4	0.362
35	115.2	713.8	0.0	24.0	0.00	13.49	10.0	0.289
36	118.5	636.5	0.0	21.4	0.00	11.21	9.5	0.264
37	121.8	678.3	0.0	22.8	0.00	10.59	8.9	0.239
38	125.0	589.2	0.0	19.8	0.00	8.39	8.3	0.215
39	128.3	638.0	0.0	21.5	0.00	7.85	7.6	0.191
40	131.6	506.4	0.0	17.0	0.00	5.71	7.0	0.172
41	134.9	493.6	0.0	16.6	0.00	5.39	7.3	0.154
42	138.2	342.3	0.0	11.5	0.00	3.28	7.5	0.140
43	141.5	392.2	0.0	13.2	0.00	1.33	7.1	0.126
Absolute	9.9			38.1			(T =	21.5 ms)
	3.3				-0.29		(T =	100.0 ms)

Analysis: 11-May-2010

GCC; Pile: P4 2nd Restrike
PP24x0.401, D62-22; Blow: 4
Robert Miner Dynamic Testing, Inc.

Test: 03-May-2010 10:53:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1426.0	1349.0	1272.1	1195.1	1118.1	1041.2	964.2	887.2	810.3	733.3
RX	1426.0	1349.0	1272.1	1195.1	1118.1	1041.2	964.2	887.2	810.3	733.3
RU	1534.9	1468.8	1402.8	1336.7	1270.6	1204.5	1138.5	1072.4	1006.3	940.2

RAU = 116.3 (kips); RA2 = 725.3 (kips)

Current CAPWAP Ru = 999.9 (kips); Corresponding J(RP)= 0.55; J(RX) = 0.55

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
20.06	21.14	1064.5	1131.1	1139.6	1.146	0.045	0.050	67.7	1359.3

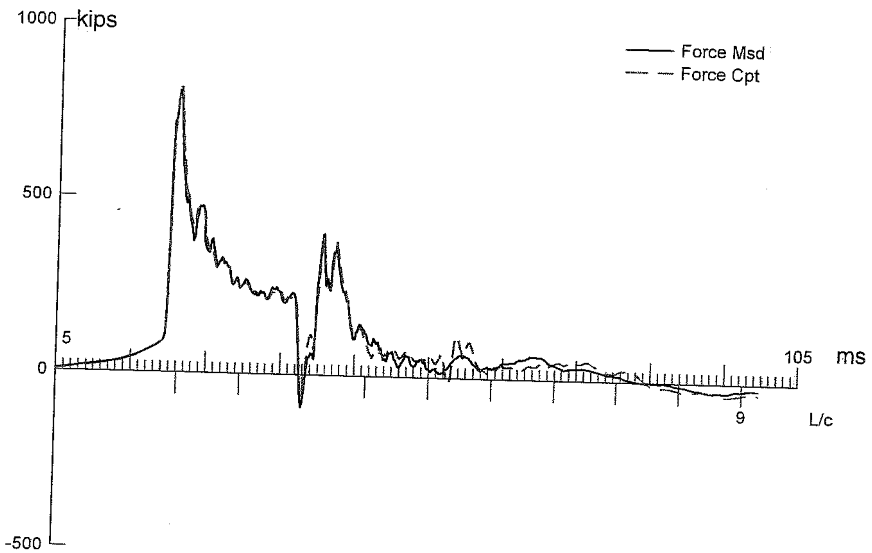
PILE PROFILE AND PILE MODEL

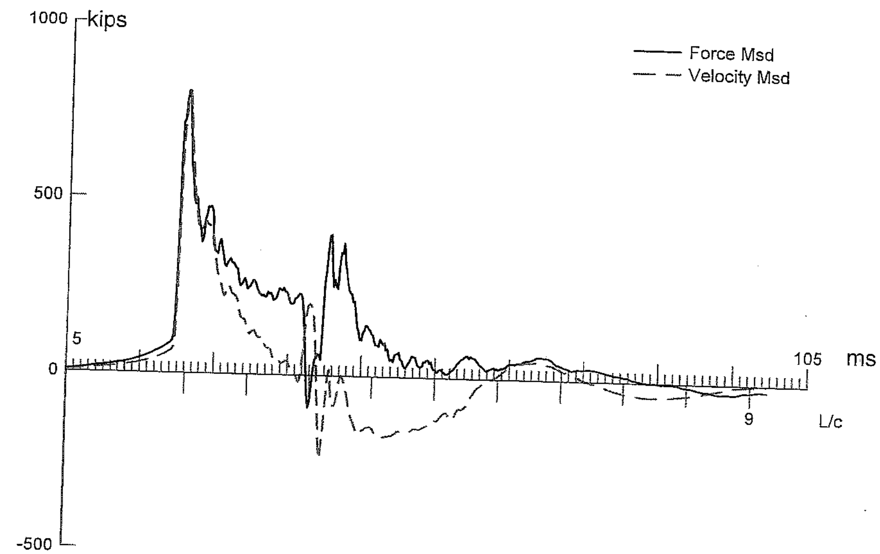
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	29.73	29992.2	492.000	6.283
141.50	29.73	29992.2	492.000	6.283

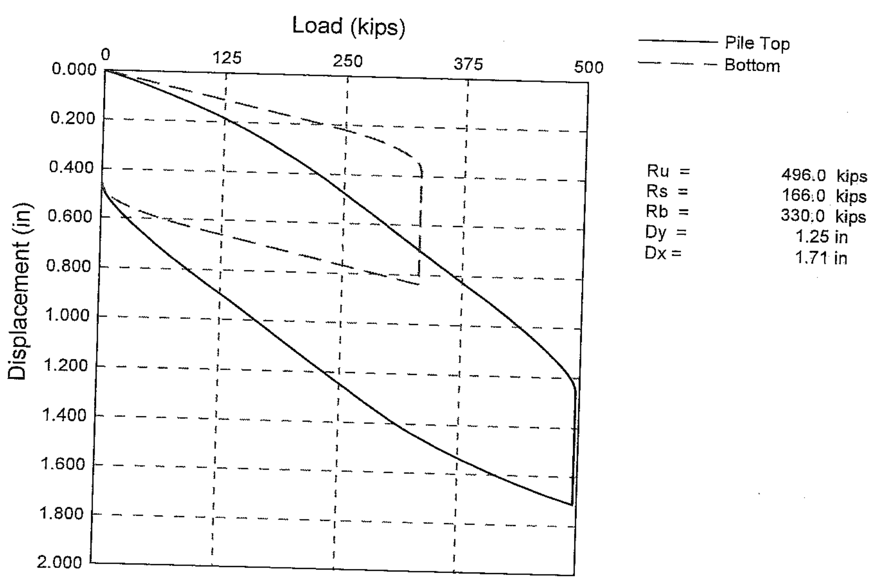
Toe Area 3.142 ft^2

Top Segment Length 3.29 ft, Top Impedance 53.06 kips/ft/s

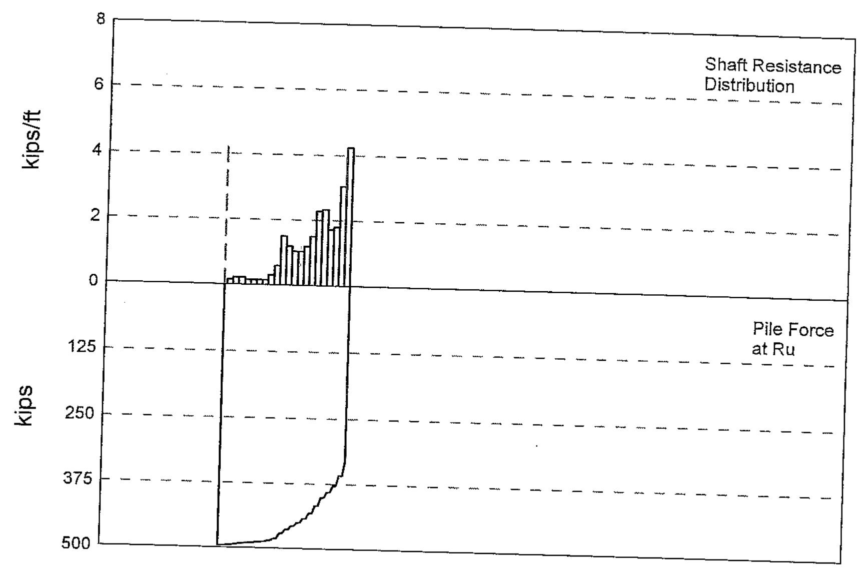
Pile Damping 1.0 %, Time Incr 0.196 ms, Wave Speed 16807.9 ft/s, 2L/c 16.8 ms







Ru = 496.0 kips
Rs = 166.0 kips
Rb = 330.0 kips
Dy = 1.25 in
Dx = 1.71 in



KIEWIT GENERAL; Pile: PILE 5, End Drive
PP24x0.401", D46-32; Blow: 749
Robert Miner Dynamic Testing, Inc.

Test: 21-Apr-2010 16:22:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 496.0; along Shaft 166.0; at Toe 330.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				496.0				
1	9.9	7.4	1.0	495.0	1.0	0.13	0.02	0.100
2	16.6	14.1	1.5	493.5	2.5	0.23	0.04	0.100
3	23.2	20.7	1.5	492.0	4.0	0.23	0.04	0.100
4	29.8	27.3	1.0	491.0	5.0	0.15	0.02	0.100
5	36.5	34.0	1.0	490.0	6.0	0.15	0.02	0.100
6	43.1	40.6	1.0	489.0	7.0	0.15	0.02	0.100
7	49.7	47.2	1.0	488.0	8.0	0.15	0.02	0.100
8	56.3	53.8	2.0	486.0	10.0	0.30	0.05	0.100
9	63.0	60.5	4.0	482.0	14.0	0.60	0.10	0.100
10	69.6	67.1	10.0	472.0	24.0	1.51	0.24	0.100
11	76.2	73.7	8.0	464.0	32.0	1.21	0.19	0.100
12	82.8	80.3	7.0	457.0	39.0	1.06	0.17	0.100
13	89.5	87.0	7.0	450.0	46.0	1.06	0.17	0.100
14	96.1	93.6	8.0	442.0	54.0	1.21	0.19	0.100
15	102.7	100.2	10.0	432.0	64.0	1.51	0.24	0.100
16	109.4	106.9	15.1	416.9	79.1	2.28	0.36	0.100
17	116.0	113.5	15.4	401.5	94.5	2.32	0.37	0.100
18	122.6	120.1	11.4	390.1	105.9	1.72	0.27	0.100
19	129.2	126.7	11.9	378.2	117.8	1.80	0.29	0.100
20	135.9	133.4	20.1	358.1	137.9	3.03	0.48	0.100
21	142.5	140.0	28.1	330.0	166.0	4.24	0.67	0.100
Avg. Shaft			7.9			1.19	0.19	0.100
Toe			330.0				1600.00	0.050

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.042	0.290
Case Damping Factor		0.313	0.311
Damping Type			Smith
Unloading Quake	(% of loading quake)	35	30
Reloading Level	(% of Ru)	100	100
Soil Plug Weight	(kips)		0.31

max. Top Comp. Stress = 27.5 ksi (T= 21.3 ms, max= 1.012 x Top)
max. Comp. Stress = 27.8 ksi (Z= 63.0 ft, T= 25.0 ms)
max. Tens. Stress = -1.58 ksi (Z= 82.8 ft, T= 62.5 ms)
max. Energy (EMX) = 36.2 kip-ft; max. Measured Top Displ. (DMX)= 1.08 in

Analysis: 11-May-2010

KIEWIT GENERAL; Pile: PILE 5, End Drive
PP24x0.401", D46-32; Blow: 749
Robert Miner Dynamic Testing, Inc.

Test: 21-Apr-2010 16:22:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	816.4	-43.8	27.5	-1.47	36.23	14.7	1.057
2	6.6	817.3	-37.9	27.5	-1.28	36.15	14.7	1.050
5	16.6	818.8	-39.9	27.5	-1.34	35.81	14.6	1.027
8	26.5	815.6	-38.6	27.4	-1.30	35.09	14.5	0.997
11	36.5	817.4	-37.8	27.5	-1.27	34.64	14.4	0.966
14	46.4	818.2	-41.4	27.5	-1.39	33.94	14.3	0.929
17	56.3	822.9	-39.2	27.7	-1.32	33.40	14.2	0.893
20	66.3	820.3	-41.6	27.6	-1.40	32.15	13.9	0.853
23	76.2	807.9	-39.1	27.2	-1.32	30.40	13.7	0.810
26	86.2	783.2	-40.6	26.3	-1.37	28.08	13.5	0.765
29	96.1	781.2	-38.2	26.3	-1.29	26.64	13.2	0.719
32	106.0	758.2	-24.7	25.5	-0.83	24.27	12.9	0.676
35	116.0	744.6	-14.1	25.0	-0.47	22.20	12.5	0.632
36	119.3	714.4	-0.6	24.0	-0.02	20.56	12.4	0.617
37	122.6	721.5	0.0	24.3	0.00	20.32	12.3	0.600
38	125.9	701.3	0.0	23.6	0.00	19.09	12.2	0.585
39	129.2	711.0	0.0	23.9	0.00	18.84	12.0	0.568
40	132.6	674.7	0.0	22.7	0.00	17.58	12.8	0.552
41	135.9	623.8	0.0	21.0	0.00	17.32	15.2	0.535
42	139.2	487.6	0.0	16.4	0.00	15.46	16.8	0.520
43	142.5	449.3	0.0	15.1	0.00	13.52	16.6	0.504
Absolute	63.0			27.8			(T =	25.0 ms)
	82.8				-1.58		(T =	62.5 ms)

Analysis: 11-May-2010

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	659.3	563.9	468.5	373.1	277.7	182.3	86.9	0.0	0.0	0.0
RX	669.3	619.1	602.3	585.4	568.6	551.8	535.0	518.1	501.3	484.5
RU	681.4	588.2	495.0	401.8	308.6	215.4	122.2	29.1	0.0	0.0

RAU = 138.5 (kips); RA2 = 351.2 (kips)

Current CAPWAP Ru = 496.0 (kips); Corresponding J(RP)= 0.17; J(RX) = 0.83

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
15.31	21.29	812.6	800.8	807.7	1.078	0.461	0.462	36.3	565.4

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	29.73	29992.2	492.000	6.283
142.50	29.73	29992.2	492.000	6.283

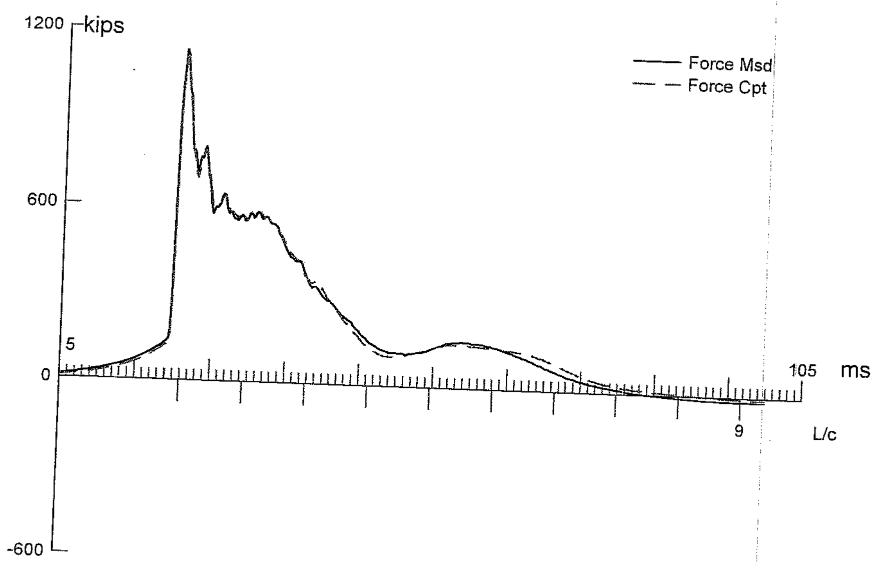
Toe Area 0.206 ft^2

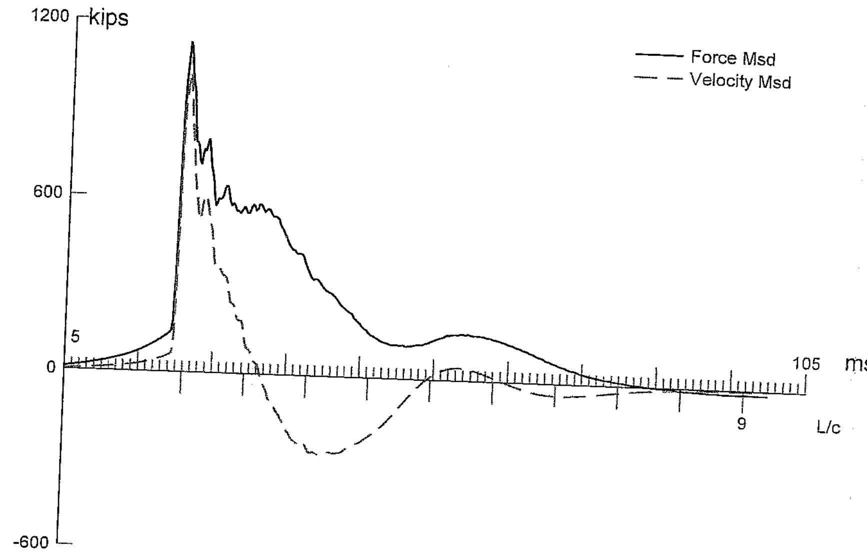
Top Segment Length 3.31 ft, Top Impedance 53.06 kips/ft/s

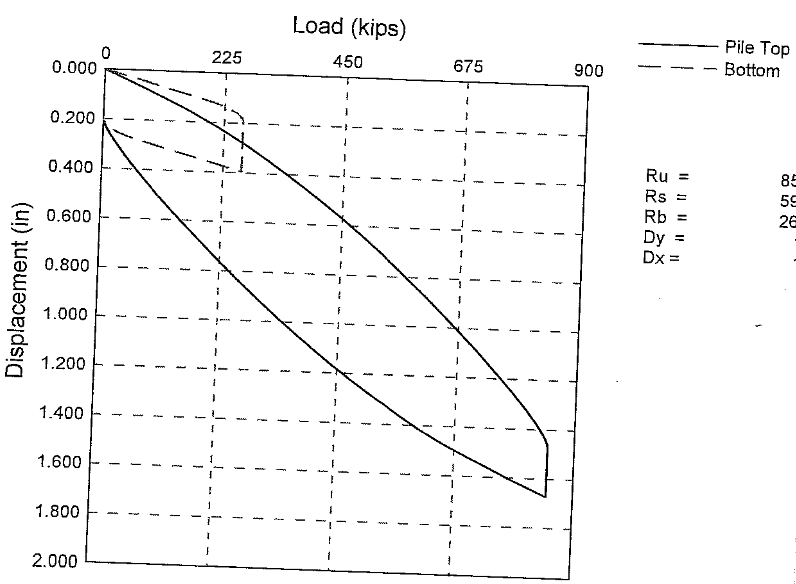
Pile Damping 1.0 %, Time Incr 0.197 ms, Wave Speed 16807.9 ft/s, 2L/c 17.0 ms

GCC,; Pile: P5 1ST RESTRKE; PP24x0.401, D62-22; Blow: 11 (Test: 26-Apr-2010 09:39:)
Robert Miner Dynamic Testing, Inc.

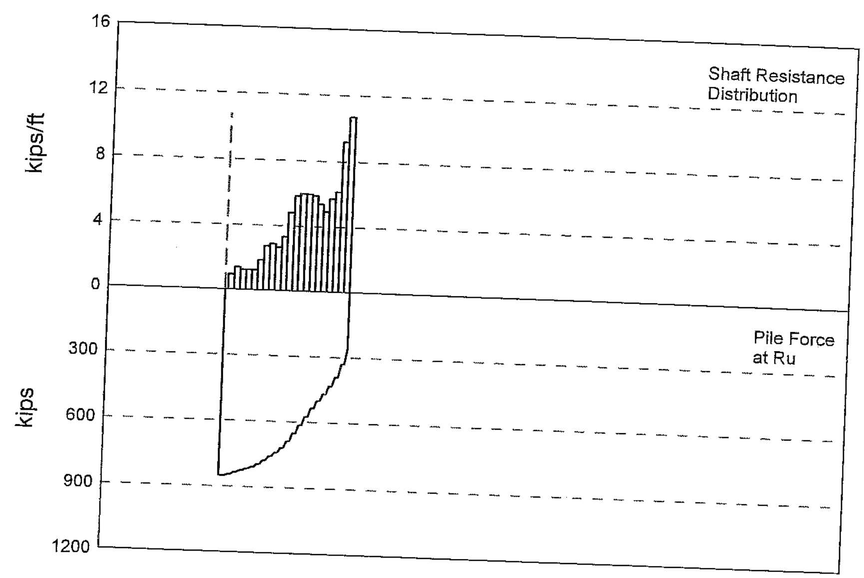
11-May-2010
CAPWAP(R) 2006-3







Ru = 850.0 kips
Rs = 590.0 kips
Rb = 260.0 kips
Dy = 1.47 in
Dx = 1.67 in



GCC,; Pile: P5 1ST RESTRKE
PP24x0.401, D62-22; Blow: 11
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 09:39:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 850.0; along Shaft 590.0; at Toe 260.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				850.0				
1	9.9	8.4	6.0	844.0	6.0	0.72	0.11	0.190
2	16.5	15.0	9.0	835.0	15.0	1.37	0.22	0.190
3	23.0	21.5	8.0	827.0	23.0	1.22	0.19	0.190
4	29.6	28.1	8.0	819.0	31.0	1.22	0.19	0.190
5	36.2	34.7	8.0	811.0	39.0	1.22	0.19	0.190
6	42.8	41.3	12.1	798.9	51.1	1.84	0.29	0.190
7	49.4	47.9	17.9	781.0	69.0	2.72	0.43	0.190
8	55.9	54.4	18.8	762.2	87.8	2.86	0.45	0.190
9	62.5	61.0	17.4	744.8	105.2	2.64	0.42	0.190
10	69.1	67.6	21.9	722.9	127.1	3.33	0.53	0.190
11	75.7	74.2	31.6	691.3	158.7	4.80	0.76	0.190
12	82.3	80.8	38.3	653.0	197.0	5.82	0.93	0.190
13	88.8	87.3	39.2	613.8	236.2	5.96	0.95	0.190
14	95.4	93.9	39.0	574.8	275.2	5.93	0.94	0.190
15	102.0	100.5	38.6	536.2	313.8	5.87	0.93	0.190
16	108.6	107.1	35.3	500.9	349.1	5.36	0.85	0.190
17	115.2	113.7	32.3	468.6	381.4	4.91	0.78	0.190
18	121.8	120.3	37.5	431.1	418.9	5.70	0.91	0.190
19	128.3	126.8	40.3	390.8	459.2	6.12	0.97	0.190
20	134.9	133.4	60.4	330.4	519.6	9.18	1.46	0.190
21	141.5	140.0	70.4	260.0	590.0	10.70	1.70	0.190
Avg. Shaft			28.1			4.21	0.67	0.190
Toe			260.0				1259.33	0.070

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.100	0.150
Case Damping Factor		2.113	0.343
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	30	
Soil Plug Weight	(kips)		0.20

max. Top Comp. Stress = 37.4 ksi (T= 21.3 ms, max= 1.016 x Top)
max. Comp. Stress = 38.0 ksi (Z= 9.9 ft, T= 21.7 ms)
max. Tens. Stress = -0.25 ksi (Z= 9.9 ft, T= 100.0 ms)
max. Energy (EMX) = 69.1 kip-ft; max. Measured Top Displ. (DMX)= 1.14 in

Analysis: 11-May-2010

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	1113.1	-6.9	37.4	-0.23	69.11	19.3	1.146
2	6.6	1119.8	-7.2	37.7	-0.24	68.46	19.1	1.120
5	16.5	1122.2	-5.3	37.7	-0.18	64.79	18.6	1.039
8	26.3	1071.1	-0.6	36.0	-0.02	58.62	18.1	0.957
11	36.2	1069.1	0.0	36.0	0.00	54.76	17.4	0.875
14	46.1	1029.2	0.0	34.6	0.00	48.84	16.6	0.794
17	55.9	1008.8	0.0	33.9	0.00	43.82	15.6	0.713
20	65.8	923.0	0.0	31.0	0.00	36.64	14.6	0.633
23	75.7	923.3	0.0	31.0	0.00	32.35	13.1	0.560
26	85.6	770.2	0.0	25.9	0.00	24.02	11.6	0.487
29	95.4	731.7	0.0	24.6	0.00	19.48	10.1	0.417
32	105.3	578.5	0.0	19.5	0.00	13.39	8.9	0.350
35	115.2	550.6	0.0	18.5	0.00	10.48	7.9	0.287
36	118.5	492.2	0.0	16.6	0.00	8.86	7.6	0.268
37	121.8	514.3	0.0	17.3	0.00	8.52	7.2	0.249
38	125.0	452.7	0.0	15.2	0.00	7.00	6.9	0.231
39	128.3	479.0	0.0	16.1	0.00	6.68	6.5	0.213
40	131.6	425.1	0.0	14.3	0.00	5.39	6.1	0.196
41	134.9	434.6	0.0	14.6	0.00	5.15	5.9	0.180
42	138.2	358.0	0.0	12.0	0.00	3.77	6.6	0.166
43	141.5	355.4	0.0	12.0	0.00	2.56	6.5	0.152
Absolute	9.9			38.0			(T =	21.7 ms)
	9.9				-0.25		(T =	100.0 ms)

GCC,; Pile: P5 1ST RESTRKE
PP24x0.401, D62-22; Blow: 11
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 09:39:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1380.8	1301.0	1221.1	1141.2	1061.4	981.5	901.7	821.8	741.9	662.1
RX	1380.8	1301.0	1221.1	1141.2	1061.4	981.5	901.7	821.8	741.9	662.1
RU	1500.2	1432.3	1364.4	1296.4	1228.5	1160.6	1092.7	1024.7	956.8	888.9

RAU = 116.2 (kips); RA2 = 730.4 (kips)

Current CAPWAP Ru = 850.0 (kips); Corresponding J(RP)= 0.66; J(RX) = 0.66

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
19.70	21.14	1045.4	1134.0	1151.4	1.141	0.200	0.200	70.0	1253.3

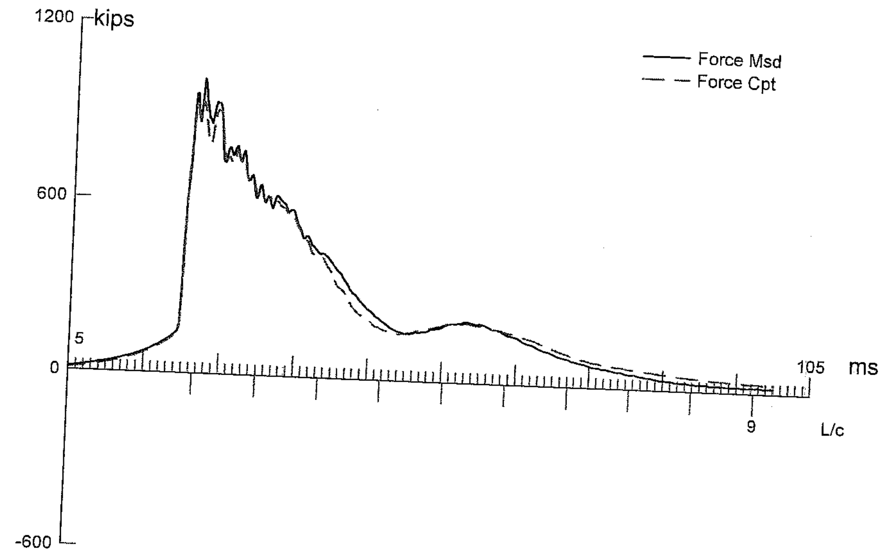
PILE PROFILE AND PILE MODEL

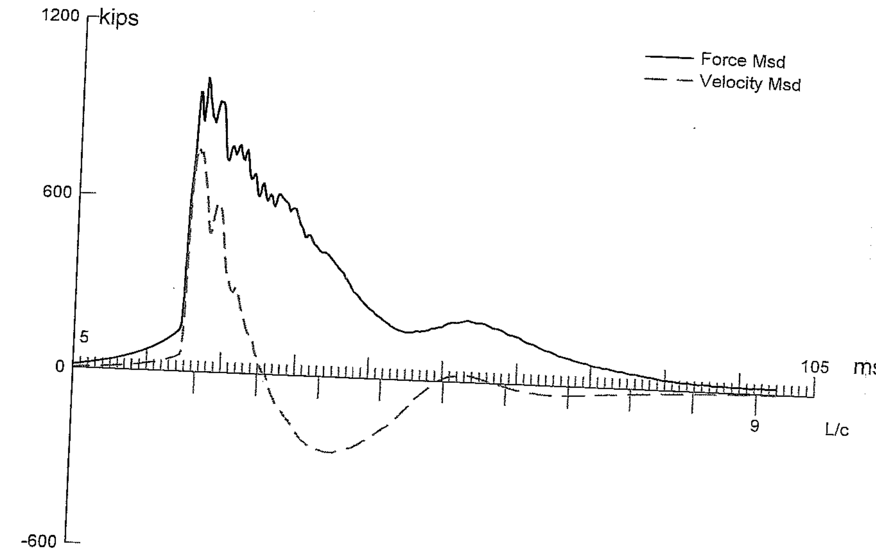
Depth ft	Area in²	E-Modulus ksi	Spec. Weight lb/ft³	Perim. ft
0.00	29.73	29992.2	492.000	6.283
141.50	29.73	29992.2	492.000	6.283

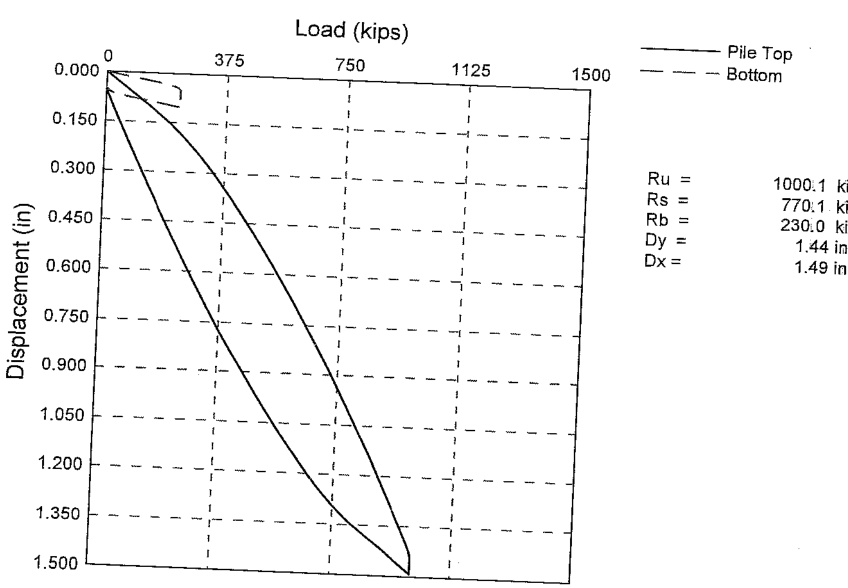
Toe Area 0.206 ft²

Top Segment Length 3.29 ft, Top Impedance 53.06 kips/ft/s

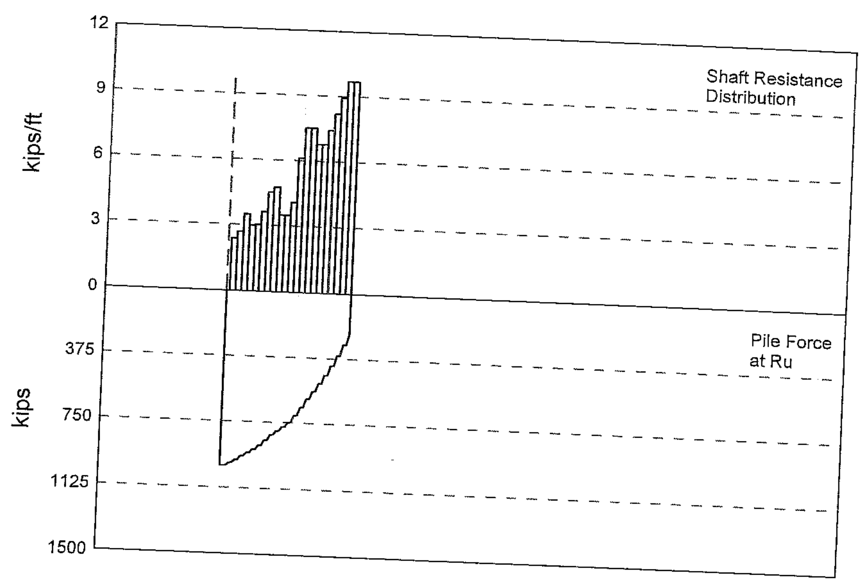
Pile Damping 1.0 %, Time Incr 0.196 ms, Wave Speed 16807.9 ft/s, 2L/c 16.8 ms







Ru = 1000.1 kips
Rs = 770.1 kips
Rb = 230.0 kips
Dy = 1.44 in
Dx = 1.49 in



GCC; Pile: Pile 5 2nd Restrike | Test: 03-May-2010 10:36:
PP24x0.401, D62-22; Blow: 14 | CAPWAP(R) 2006-3
Robert Miner Dynamic Testing, Inc. | OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 1000.1; along Shaft 770.1; at Toe 230.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft	Quake in
				1000.1					
1	9.9	8.4	15.6	984.5	15.6	1.86	0.30	0.170	0.100
2	16.5	15.0	17.6	966.9	33.2	2.67	0.43	0.170	0.100
3	23.0	21.5	22.8	944.1	56.0	3.46	0.55	0.170	0.100
4	29.6	28.1	19.6	924.5	75.6	2.98	0.47	0.170	0.100
5	36.2	34.7	19.8	904.7	95.4	3.01	0.48	0.170	0.100
6	42.8	41.3	23.6	881.1	119.0	3.59	0.57	0.170	0.100
7	49.4	47.9	29.7	851.4	148.7	4.51	0.72	0.170	0.100
8	55.9	54.4	31.3	820.1	180.0	4.76	0.76	0.170	0.100
9	62.5	61.0	22.8	797.3	202.8	3.46	0.55	0.170	0.100
10	69.1	67.6	22.8	774.5	225.6	3.46	0.55	0.170	0.100
11	75.7	74.2	26.7	747.8	252.3	4.06	0.65	0.170	0.100
12	82.3	80.8	40.2	707.6	292.5	6.11	0.97	0.170	0.100
13	88.8	87.3	49.5	658.1	342.0	7.52	1.20	0.170	0.095
14	95.4	93.9	49.5	608.6	391.5	7.52	1.20	0.170	0.090
15	102.0	100.5	44.6	564.0	436.1	6.78	1.08	0.170	0.085
16	108.6	107.1	44.6	519.4	480.7	6.78	1.08	0.170	0.080
17	115.2	113.7	49.0	470.4	529.7	7.45	1.18	0.170	0.070
18	121.8	120.3	54.0	416.4	583.7	8.20	1.31	0.170	0.060
19	128.3	126.8	58.8	357.6	642.5	8.93	1.42	0.170	0.050
20	134.9	133.4	63.8	293.8	706.3	9.69	1.54	0.170	0.040
21	141.5	140.0	63.8	230.0	770.1	9.69	1.54	0.170	0.040
Avg. Shaft			36.7			5.50	0.88	0.170	0.079
Toe			230.0				1114.03	0.035	0.040

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		2.467	0.152
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	0	

max. Top Comp. Stress = 31.8 ksi (T= 22.3 ms, max= 1.031 x Top)
max. Comp. Stress = 32.7 ksi (Z= 9.9 ft, T= 22.7 ms)
max. Tens. Stress = 0.00 ksi (Z= 3.3 ft, T= 0.0 ms)
max. Energy (EMX) = 66.3 kip-ft; max. Measured Top Displ. (DMX)= 1.02 in

GCC; Pile: Pile 5 2nd Restrike
PP24x0.401, D62-22; Blow: 14
Robert Miner Dynamic Testing, Inc.

Test: 03-May-2010 10:36:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	944.2	0.0	31.8	0.00	66.30	15.1	1.056
2	6.6	956.9	0.0	32.2	0.00	65.58	14.9	1.028
5	16.5	949.0	0.0	31.9	0.00	60.14	14.0	0.943
8	26.3	860.9	0.0	28.9	0.00	50.62	13.2	0.858
11	36.2	850.1	0.0	28.6	0.00	45.29	12.3	0.773
14	46.1	776.5	0.0	26.1	0.00	37.30	11.3	0.689
17	55.9	760.3	0.0	25.6	0.00	31.80	10.3	0.607
20	65.8	697.7	0.0	23.5	0.00	24.62	9.5	0.526
23	75.7	697.4	0.0	23.5	0.00	21.04	8.5	0.450
26	85.6	620.0	0.0	20.9	0.00	15.27	7.4	0.373
29	95.4	585.1	0.0	19.7	0.00	11.24	6.2	0.295
32	105.3	500.5	0.0	16.8	0.00	6.80	5.4	0.228
35	115.2	511.4	0.0	17.2	0.00	4.58	4.4	0.165
36	118.5	472.3	0.0	15.9	0.00	3.47	4.2	0.147
37	121.8	465.1	0.0	15.6	0.00	3.14	3.8	0.128
38	125.0	411.3	0.0	13.8	0.00	2.24	3.6	0.111
39	128.3	423.4	0.0	14.2	0.00	1.96	3.2	0.094
40	131.6	362.7	0.0	12.2	0.00	1.34	3.2	0.081
41	134.9	354.8	0.0	11.9	0.00	1.17	3.4	0.067
42	138.2	285.1	0.0	9.6	0.00	0.71	3.3	0.055
43	141.5	300.3	0.0	10.1	0.00	0.48	2.9	0.042
Absolute	9.9			32.7			(T =	22.7 ms)
	3.3				0.00		(T =	0.0 ms)

Analysis: 12-May-2010

GCC; Pile: Pile 5 2nd Restrike
PP24x0.401, D62-22; Blow: 14
Robert Miner Dynamic Testing, Inc.

Test: 03-May-2010 10:36:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1204.4	1152.4	1100.4	1048.4	996.4	944.4	892.4	840.4	788.3	736.3
RX	1219.1	1165.1	1111.2	1057.2	1003.2	949.2	895.2	841.2	788.3	736.3
RU	1390.3	1356.9	1323.5	1290.1	1256.7	1223.2	1189.8	1156.4	1123.0	1089.6

RAU = 179.3 (kips); RA2 = 1019.3 (kips)

Current CAPWAP Ru = 1000.1 (kips); Corresponding J(RP)= 0.39; J(RX) = 0.41

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
14.57	21.73	773.1	951.4	1025.7	1.020	0.047	0.050	65.9	1477.9

PILE PROFILE AND PILE MODEL

Depth ft	Area in²	E-Modulus ksi	Spec. Weight lb/ft³	Perim. ft
0.00	29.73	29992.2	492.000	6.283
141.50	29.73	29992.2	492.000	6.283

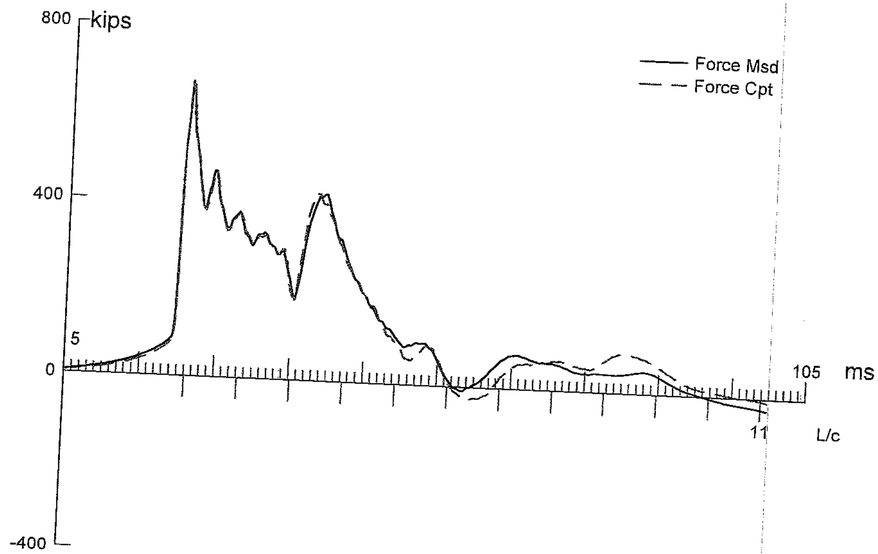
Toe Area 0.206 ft²

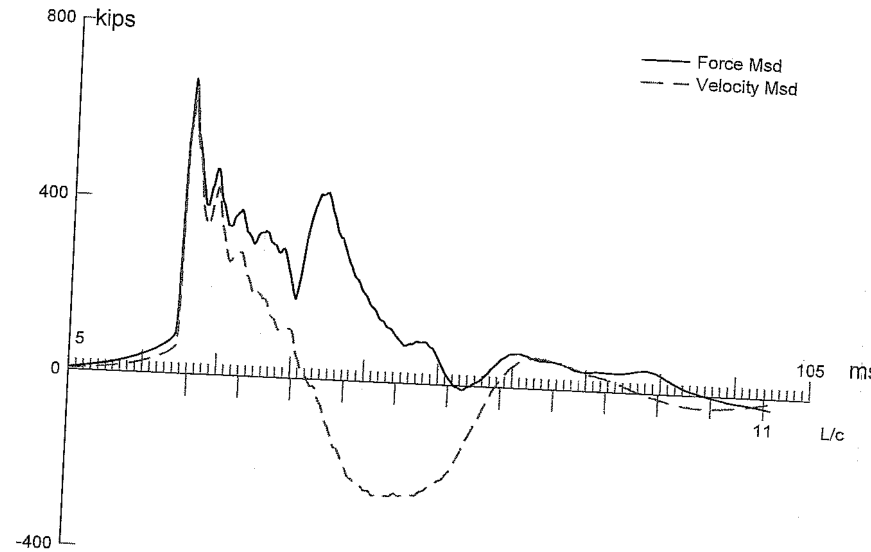
Top Segment Length 3.29 ft, Top Impedance 53.06 kips/ft/s

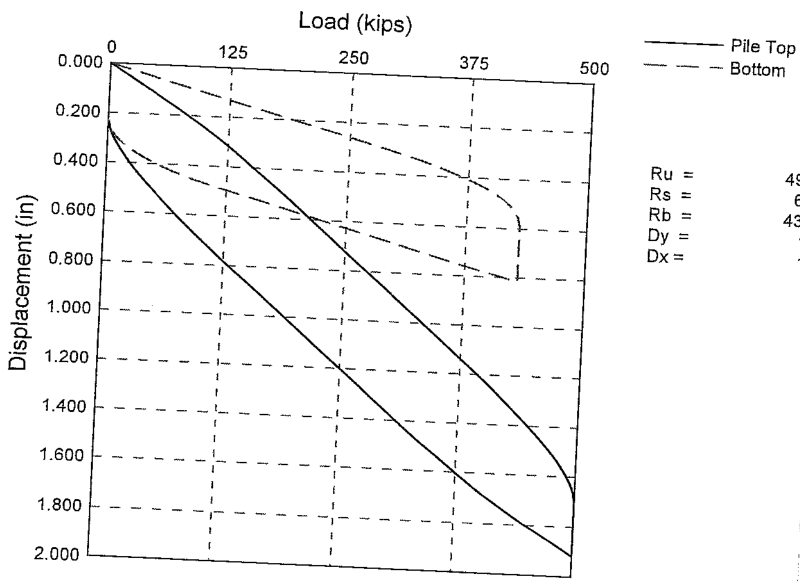
Pile Damping 1.0 %, Time Incr 0.196 ms, Wave Speed 16807.9 ft/s, 2L/c 16.8 ms

KIEWIT GENERAL,; Pile: PILE 6, END DRIVE; PP18x0.375", D46-32; Blow: 679 (Test: 21-Apr-2010 13:46:)
Robert Miner Dynamic Testing, Inc.

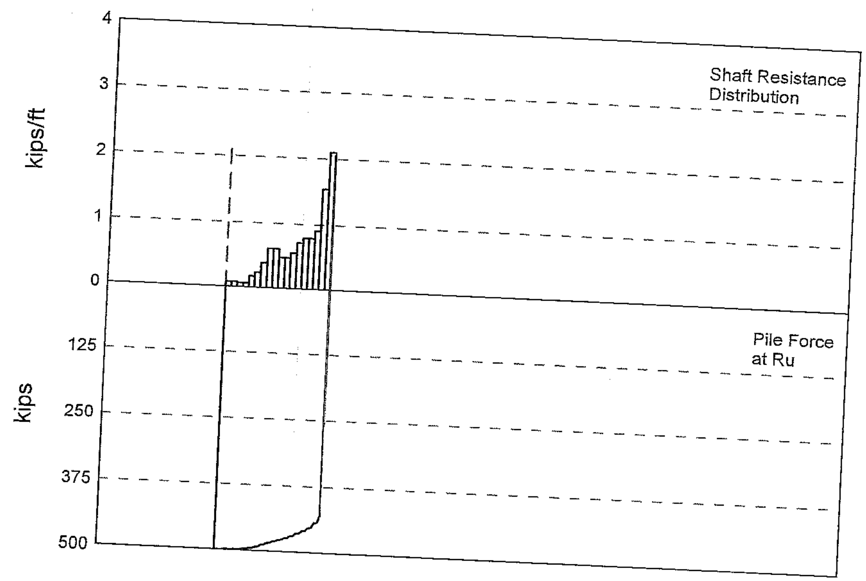
12-May-2010
CAPWAP(R) 2006-3







Ru =	499.9	kips
Rs =	69.9	kips
Rb =	430.0	kips
Dy =	1.69	in
Dx =	1.92	in



KIEWIT GENERAL,; Pile: PILE 6, END DRIVE
PP18x0.375", D46-32; Blow: 679
Robert Miner Dynamic Testing, Inc.

Test: 21-Apr-2010 13:46:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 499.9; along Shaft 69.9; at Toe 430.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				499.9				
1	6.6	5.1	0.5	499.4	0.5	0.10	0.02	0.221
2	13.2	11.7	0.5	498.9	1.0	0.08	0.02	0.221
3	19.8	18.3	0.4	498.5	1.4	0.06	0.01	0.221
4	26.4	24.9	0.4	498.1	1.8	0.06	0.01	0.221
5	33.1	31.6	1.1	497.0	2.9	0.17	0.04	0.221
6	39.7	38.2	1.5	495.5	4.4	0.23	0.05	0.221
7	46.3	44.8	2.5	493.0	6.9	0.38	0.08	0.221
8	52.9	51.4	4.0	489.0	10.9	0.61	0.13	0.221
9	59.5	58.0	4.0	485.0	14.9	0.61	0.13	0.221
10	66.1	64.6	3.1	481.9	18.0	0.47	0.10	0.221
11	72.7	71.2	3.1	478.8	21.1	0.47	0.10	0.221
12	79.3	77.8	3.6	475.2	24.7	0.54	0.12	0.221
13	85.9	84.4	4.7	470.5	29.4	0.71	0.15	0.221
14	92.6	91.1	5.2	465.3	34.6	0.79	0.17	0.221
15	99.2	97.7	5.2	460.1	39.8	0.79	0.17	0.221
16	105.8	104.3	6.0	454.1	45.8	0.91	0.19	0.221
17	112.4	110.9	10.2	443.9	56.0	1.54	0.33	0.221
18	119.0	117.5	13.9	430.0	69.9	2.10	0.45	0.221
Avg. Shaft			3.9			0.59	0.13	0.221
Toe			430.0				243.33	0.039

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.123	0.450
Case Damping Factor		0.418	0.454
Unloading Quake	(% of loading quake)	30	30
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	50	
Soil Plug Weight	(kips)		0.03

max. Top Comp. Stress = 32.0 ksi (T= 21.2 ms, max= 1.010 x Top)
max. Comp. Stress = 32.4 ksi (Z= 46.3 ft, T= 24.0 ms)
max. Tens. Stress = -5.39 ksi (Z= 39.7 ft, T= 59.8 ms)
max. Energy (EMX) = 49.8 kip-ft; max. Measured Top Displ. (DMX)= 1.54 in

Analysis: 12-May-2010

KIEWIT GENERAL,; Pile: PILE 6, END DRIVE
PP18x0.375", D46-32; Blow: 679
Robert Miner Dynamic Testing, Inc.

Test: 21-Apr-2010 13:46:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	665.6	-35.1	32.0	-1.69	49.77	17.4	1.544
2	6.6	666.5	-45.4	32.1	-2.19	49.59	17.3	1.528
4	13.2	666.5	-62.1	32.1	-2.99	48.92	17.2	1.490
6	19.8	666.5	-78.2	32.1	-3.76	48.09	17.2	1.445
8	26.4	668.0	-91.9	32.2	-4.43	47.15	17.0	1.396
10	33.1	670.4	-102.6	32.3	-4.94	46.16	16.9	1.345
12	39.7	670.9	-112.1	32.3	-5.39	44.87	16.7	1.291
14	46.3	671.9	-110.3	32.4	-5.31	43.37	16.5	1.235
16	52.9	668.8	-107.6	32.2	-5.18	41.53	16.3	1.177
18	59.5	658.0	-102.7	31.7	-4.94	39.39	16.0	1.121
20	66.1	646.8	-97.4	31.1	-4.69	37.27	15.8	1.064
22	72.7	640.5	-92.6	30.8	-4.46	35.35	15.6	1.005
24	79.3	635.9	-90.4	30.6	-4.35	33.43	15.3	0.945
26	85.9	630.4	-87.8	30.4	-4.23	31.51	15.0	0.886
28	92.6	620.6	-83.0	29.9	-4.00	29.45	14.7	0.828
30	99.2	609.9	-77.8	29.4	-3.75	27.33	14.3	0.770
31	102.5	594.4	-72.9	28.6	-3.51	25.86	14.1	0.740
32	105.8	603.1	-72.5	29.0	-3.49	25.28	13.9	0.710
33	109.1	572.0	-65.4	27.5	-3.15	23.77	14.1	0.681
34	112.4	572.8	-64.5	27.6	-3.11	23.24	15.3	0.652
35	115.7	551.2	-53.6	26.5	-2.58	21.26	15.6	0.624
36	119.0	542.9	-52.5	26.1	-2.53	19.71	15.0	0.595
Absolute	46.3			32.4			(T =	24.0 ms)
	39.7				-5.39		(T =	59.8 ms)

Analysis: 12-May-2010

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	696.8	636.3	575.9	515.4	455.0	394.6	334.1	273.7	213.2	152.8
RX	715.0	654.5	632.4	610.2	588.0	565.9	544.3	523.8	510.4	498.6
RU	696.8	636.3	575.9	515.4	455.0	394.6	334.1	273.7	213.2	152.8

RAU = 400.9 (kips); RA2 = 540.9 (kips)

Current CAPWAP Ru = 499.9 (kips); Corresponding J(RP)= 0.33; J(RX) = 0.89

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
17.96	21.24	627.7	673.5	686.7	1.540	0.213	0.231	50.2	680.2

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	20.76	29992.2	492.000	4.712
119.00	20.76	29992.2	492.000	4.712

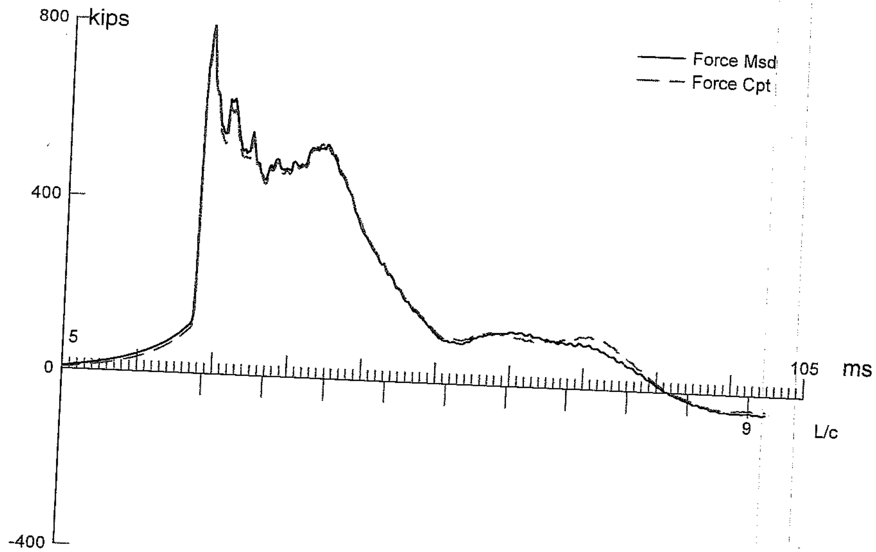
Toe Area 1.767 ft^2

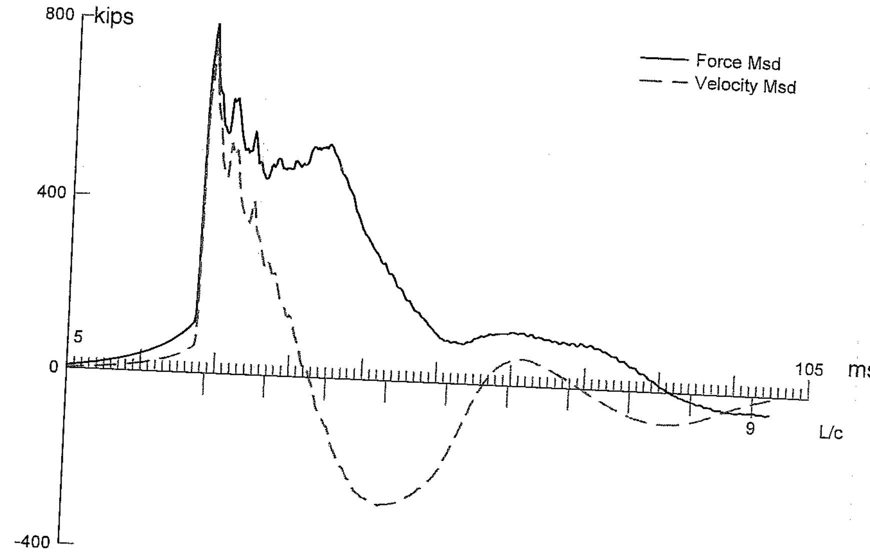
Top Segment Length 3.31 ft, Top Impedance 37.06 kips/ft/s

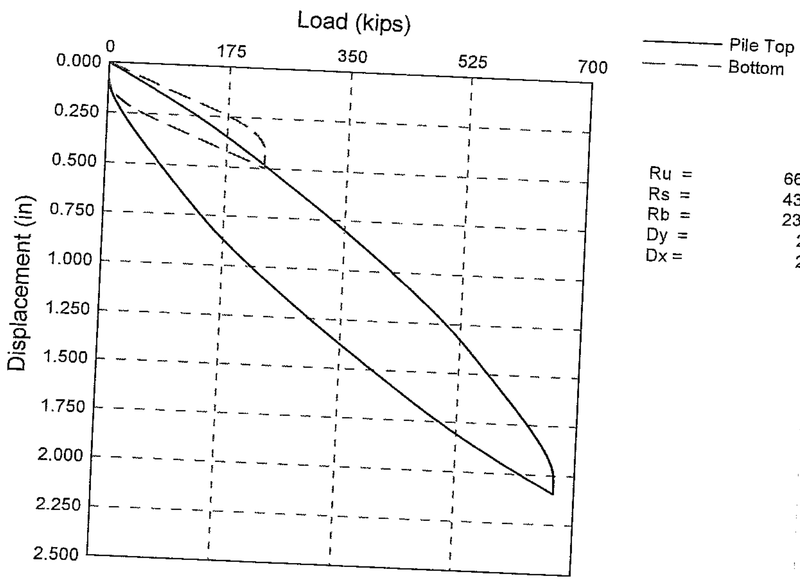
Pile Damping 2.0 %, Time Incr 0.197 ms, Wave Speed 16807.9 ft/s, 2L/c 14.2 ms

GCC; Pile: P6 1ST RESTRKE; PP18x0.375, D62-22; Blow: 5 (Test: 26-Apr-2010 08:47:)
Robert Miner Dynamic Testing, Inc.

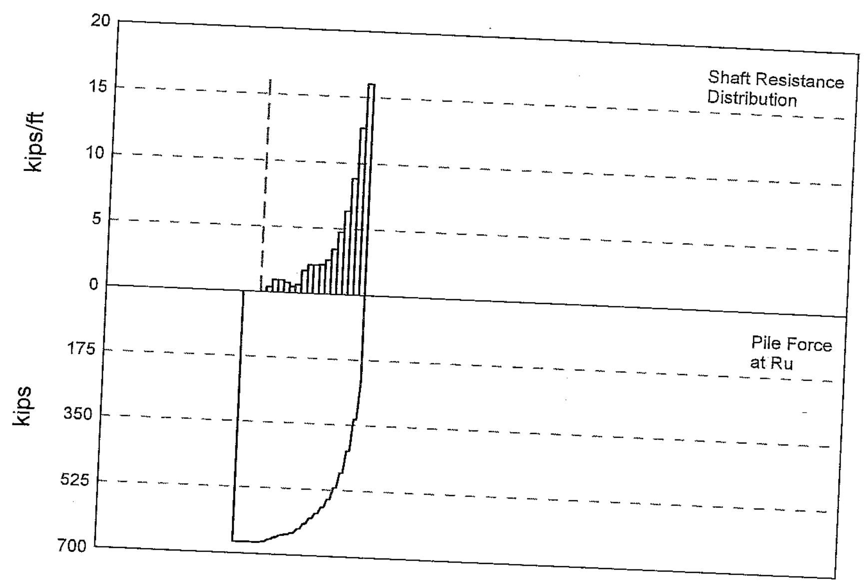
12-May-2010
CAPWAP(R) 2006-3







Ru =	669.9 kips
Rs =	439.9 kips
Rb =	230.0 kips
Dy =	2.01 in
Dx =	2.09 in



GCC; Pile: P6 1ST RESTRKE
PP18x0.375, D62-22; Blow: 5
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 08:47:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 669.9; along Shaft 439.9; at Toe 230.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				669.9				
1	26.3	5.8	0.0	669.9	0.0	0.00	0.00	0.000
2	32.9	12.4	2.6	667.3	2.6	0.40	0.08	0.190
3	39.4	18.9	6.1	661.2	8.7	0.93	0.20	0.190
4	46.0	25.5	6.1	655.1	14.8	0.93	0.20	0.190
5	52.6	32.1	5.1	650.0	19.9	0.78	0.16	0.190
6	59.1	38.6	3.1	646.9	23.0	0.47	0.10	0.190
7	65.7	45.2	4.1	642.8	27.1	0.62	0.13	0.190
8	72.3	51.8	11.3	631.5	38.4	1.72	0.36	0.190
9	78.9	58.4	14.0	617.5	52.4	2.13	0.45	0.190
10	85.4	64.9	14.1	603.4	66.5	2.15	0.46	0.190
11	92.0	71.5	14.5	588.9	81.0	2.21	0.47	0.190
12	98.6	78.1	16.9	572.0	97.9	2.57	0.55	0.190
13	105.1	84.6	22.4	549.6	120.3	3.41	0.72	0.190
14	111.7	91.2	31.1	518.5	151.4	4.73	1.00	0.190
15	118.3	97.8	41.7	476.8	193.1	6.35	1.35	0.190
16	124.9	104.4	58.2	418.6	251.3	8.86	1.88	0.190
17	131.4	110.9	83.3	335.3	334.6	12.68	2.69	0.190
18	138.0	117.5	105.3	230.0	439.9	16.02	3.40	0.190
Avg. Shaft			24.4			3.74	0.79	0.190
Toe			230.0				130.15	0.050

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.100	0.320
Case Damping Factor		2.255	0.310
Unloading Quake	(% of loading quake)	56	90
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	25	
Soil Plug Weight	(kips)		0.03

max. Top Comp. Stress = 38.3 ksi (T= 24.0 ms, max= 1.029 x Top)
max. Comp. Stress = 39.4 ksi (Z= 39.4 ft, T= 26.2 ms)
max. Tens. Stress = -2.23 ksi (Z= 39.4 ft, T= 96.4 ms)
max. Energy (EMX) = 76.6 kip-ft; max. Measured Top Displ. (DMX)= 1.72 in

Analysis: 12-May-2010

GCC; Pile: P6 1ST RESTRKE
PP18x0.375, D62-22; Blow: 5
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 08:47:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	795.8	-36.8	38.3	-1.77	76.56	20.2	1.764
2	6.6	796.9	-37.9	38.4	-1.82	75.76	20.2	1.730
4	13.1	799.1	-40.0	38.5	-1.93	74.19	20.1	1.661
6	19.7	801.6	-41.9	38.6	-2.02	72.56	20.0	1.590
8	26.3	807.3	-44.0	38.9	-2.12	70.78	19.8	1.516
10	32.9	818.4	-45.6	39.4	-2.20	68.85	19.5	1.440
12	39.4	818.9	-46.4	39.4	-2.23	65.98	19.1	1.363
14	46.0	802.4	-45.6	38.6	-2.20	61.91	18.7	1.284
16	52.6	783.9	-44.8	37.7	-2.16	58.03	18.5	1.207
18	59.1	770.6	-44.4	37.1	-2.14	54.51	18.2	1.128
20	65.7	774.9	-44.6	37.3	-2.15	51.61	17.6	1.049
22	72.3	781.0	-44.4	37.6	-2.14	48.54	17.0	0.969
24	78.9	758.6	-41.6	36.5	-2.00	44.06	16.3	0.891
26	85.4	727.0	-37.8	35.0	-1.82	39.49	15.5	0.816
28	92.0	698.8	-34.1	33.6	-1.64	35.28	14.8	0.742
30	98.6	693.6	-30.3	33.4	-1.46	31.42	13.9	0.670
32	105.1	668.0	-25.2	32.2	-1.21	27.71	12.8	0.601
34	111.7	646.5	-18.8	31.1	-0.91	23.89	11.4	0.535
36	118.3	610.7	-9.8	29.4	-0.47	19.90	9.8	0.472
38	124.9	556.9	0.0	26.8	0.00	15.96	8.0	0.414
40	131.4	474.6	0.0	22.8	0.00	12.02	7.1	0.364
42	138.0	367.7	0.0	17.7	0.00	4.10	6.2	0.321
Absolute	39.4			39.4			(T = 26.2 ms)	
	39.4				-2.23		(T = 96.4 ms)	

Analysis: 12-May-2010

Test: 26-Apr-2010 08:47:
GCC; Pile: P6 1ST RESTRKE CAPWAP(R) 2006-3
PP18x0.375, D62-22; Blow: 5 OP: RMDT:--RMINER
Robert Miner Dynamic Testing, Inc.

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1031.4	990.5	949.6	908.6	867.7	826.8	785.8	744.9	704.0	663.1
RX	1093.5	1045.9	998.4	950.8	903.2	855.6	808.0	760.4	713.0	668.2
RU	1093.6	1051.3	1009.0	966.7	924.4	882.1	839.8	797.5	755.2	712.9

RAU = 137.4 (kips); RA2 = 743.5 (kips)

Current CAPWAP Ru = 669.9 (kips); Corresponding J(RP)= 0.88; J(RX) = 0.90

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
20.61	23.85	685.5	755.2	807.0	1.716	0.083	0.083	76.8	1024.2

PILE PROFILE AND PILE MODEL

Depth ft	Area in²	E-Modulus ksi	Spec. Weight lb/ft³	Perim. ft
0.00	20.76	29992.2	492.000	4.712
138.00	20.76	29992.2	492.000	4.712

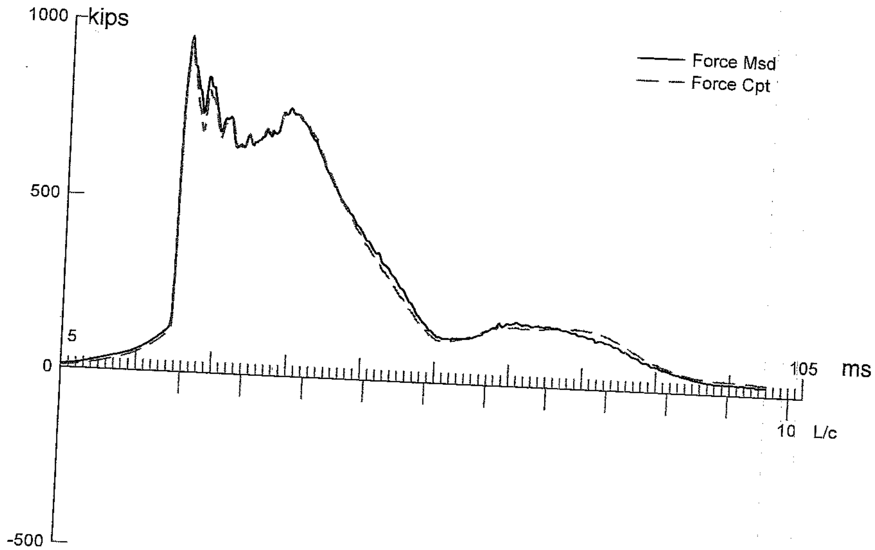
Toe Area 1.767 ft²

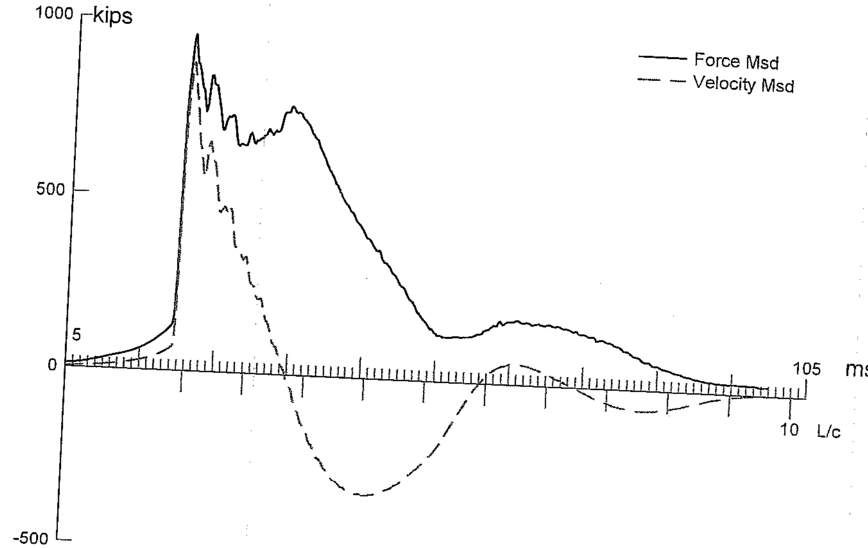
Top Segment Length 3.29 ft, Top Impedance 37.06 kips/ft/s

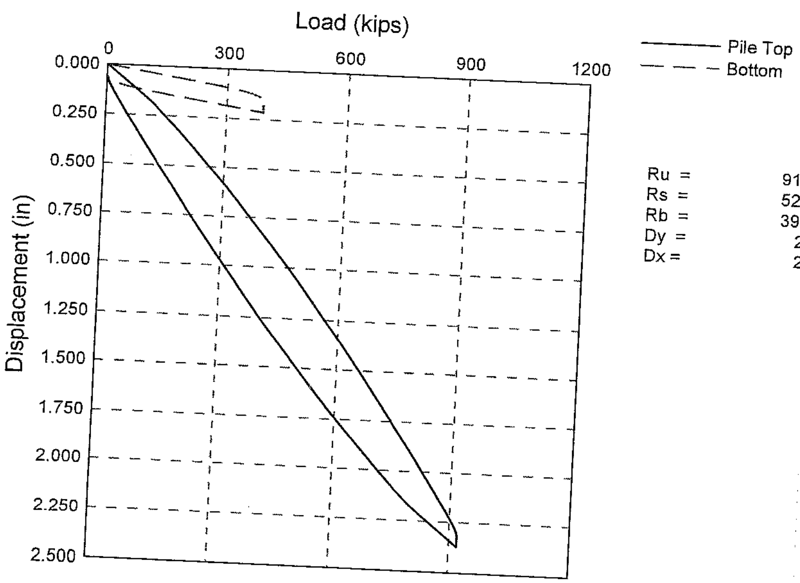
Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 16807.9 ft/s, 2L/c 16.4 ms

GCC; Pile: PILE 6 2ND RESTRIKE; PP18X0.375", D62-22; Blow: 10 (Test: 03-May-2010 09:07:)
Robert Miner Dynamic Testing, Inc.

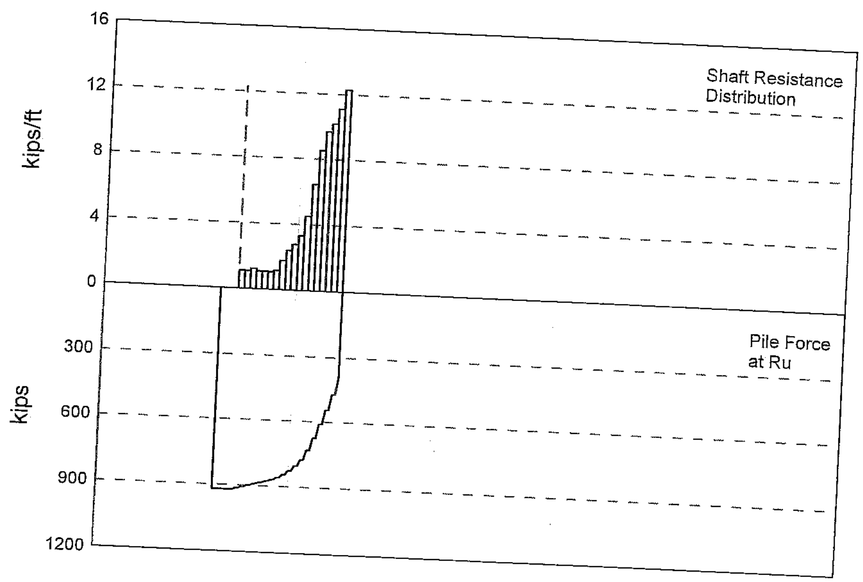
12-May-2010
CAPWAP(R) 2006-3







Ru = 919.9 kips
Rs = 529.9 kips
Rb = 390.0 kips
Dy = 2.31 in
Dx = 2.37 in



GCC; Pile: PILE 6 2ND RESTRIKE
PP18X0.375", D62-22; Blow: 10
Robert Miner Dynamic Testing, Inc.

Test: 03-May-2010 09:07:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 919.9; along Shaft 529.9; at Toe 390.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft	Quake in
				919.9					
1	26.3	6.3	7.0	912.9	7.0	1.11	0.24	0.260	0.100
2	32.9	12.9	7.0	905.9	14.0	1.07	0.23	0.260	0.100
3	39.4	19.4	8.0	897.9	22.0	1.22	0.26	0.260	0.100
4	46.0	26.0	7.0	890.9	29.0	1.07	0.23	0.260	0.100
5	52.6	32.6	7.0	883.9	36.0	1.07	0.23	0.260	0.100
6	59.1	39.1	6.8	877.1	42.8	1.03	0.22	0.260	0.100
7	65.7	45.7	7.5	869.6	50.3	1.14	0.24	0.260	0.100
8	72.3	52.3	11.4	858.2	61.7	1.73	0.37	0.260	0.100
9	78.9	58.9	15.5	842.7	77.2	2.36	0.50	0.260	0.100
10	85.4	65.4	18.0	824.7	95.2	2.74	0.58	0.260	0.100
11	92.0	72.0	21.6	803.1	116.8	3.29	0.70	0.260	0.100
12	98.6	78.6	29.6	773.5	146.4	4.50	0.96	0.260	0.100
13	105.1	85.1	42.5	731.0	188.9	6.47	1.37	0.260	0.100
14	111.7	91.7	56.1	674.9	245.0	8.54	1.81	0.260	0.100
15	118.3	98.3	63.9	611.0	308.9	9.72	2.06	0.260	0.100
16	124.9	104.9	67.1	543.9	376.0	10.21	2.17	0.260	0.100
17	131.4	111.4	73.1	470.8	449.1	11.12	2.36	0.260	0.070
18	138.0	118.0	80.8	390.0	529.9	12.30	2.61	0.260	0.021
Avg. Shaft			29.4			4.49	0.95	0.260	0.084
Toe			390.0				220.69	0.160	0.120

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		3.717	1.684
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	10	
Soil Plug Weight	(kips)		0.15

max. Top Comp. Stress = 45.6 ksi (T= 21.3 ms, max= 1.048 x Top)
max. Comp. Stress = 47.8 ksi (Z= 26.3 ft, T= 22.7 ms)
max. Tens. Stress = 0.00 ksi (Z= 3.3 ft, T= 0.0 ms)
max. Energy (EMX) = 121.3 kip-ft; max. Measured Top Displ. (DMX)= 2.03 in

Analysis: 12-May-2010

GCC; Pile: PILE 6 2ND RESTRIKE
PP18X0.375", D62-22; Blow: 10
Robert Miner Dynamic Testing, Inc.

Test: 03-May-2010 09:07:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	947.3	0.0	45.6	0.00	121.32	24.0	2.072
2	6.6	949.2	0.0	45.7	0.00	119.71	23.9	2.022
4	13.1	952.9	0.0	45.9	0.00	116.43	23.8	1.922
6	19.7	970.4	0.0	46.7	0.00	113.04	23.3	1.820
8	26.3	992.8	0.0	47.8	0.00	109.53	22.7	1.717
10	32.9	968.4	0.0	46.6	0.00	101.72	22.0	1.615
12	39.4	943.9	0.0	45.4	0.00	94.09	21.4	1.511
14	46.0	913.1	0.0	44.0	0.00	86.26	20.8	1.406
16	52.6	889.5	0.0	42.8	0.00	79.21	20.2	1.301
18	59.1	867.6	0.0	41.8	0.00	72.51	19.6	1.195
20	65.7	865.2	0.0	41.7	0.00	66.25	18.8	1.091
22	72.3	872.1	0.0	42.0	0.00	60.12	17.8	0.986
24	78.9	867.9	0.0	41.8	0.00	53.28	16.7	0.883
26	85.4	863.8	0.0	41.6	0.00	46.08	15.5	0.783
28	92.0	849.6	0.0	40.9	0.00	39.20	14.0	0.684
30	98.6	822.7	0.0	39.6	0.00	32.64	12.2	0.588
32	105.1	785.5	0.0	37.8	0.00	26.13	10.3	0.496
34	111.7	736.9	0.0	35.5	0.00	19.73	8.3	0.409
36	118.3	674.0	0.0	32.4	0.00	13.98	6.6	0.327
38	124.9	605.6	0.0	29.2	0.00	9.42	5.2	0.252
40	131.4	546.5	0.0	26.3	0.00	6.08	3.7	0.184
42	138.0	488.2	0.0	23.5	0.00	2.79	2.1	0.123
Absolute	26.3			47.8			(T =	22.7 ms)
	3.3				0.00		(T =	0.0 ms)

Analysis: 12-May-2010

Test: 03-May-2010 09:07:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

GCC; Pile: PILE 6 2ND RESTRIKE
PP18X0.375", D62-22; Blow: 10
Robert Miner Dynamic Testing, Inc.

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1339.8	1288.5	1237.2	1186.0	1134.7	1083.5	1032.2	980.9	929.7	878.4
RX	1340.6	1289.4	1238.2	1187.0	1135.8	1084.5	1033.3	982.1	930.9	880.2
RU	1405.5	1360.8	1316.1	1271.4	1226.7	1182.0	1137.3	1092.6	1047.9	1003.2

RAU = 248.1 (kips); RA2 = 1027.7 (kips)

Current CAPWAP Ru = 919.9 (kips); Corresponding J(RP)= 0.82; J(RX) = 0.82

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
23.95	21.11	887.5	964.9	966.0	2.032	0.053	0.059	122.7	1407.7

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	20.76	29992.2	492.000	4.712
138.00	20.76	29992.2	492.000	4.712

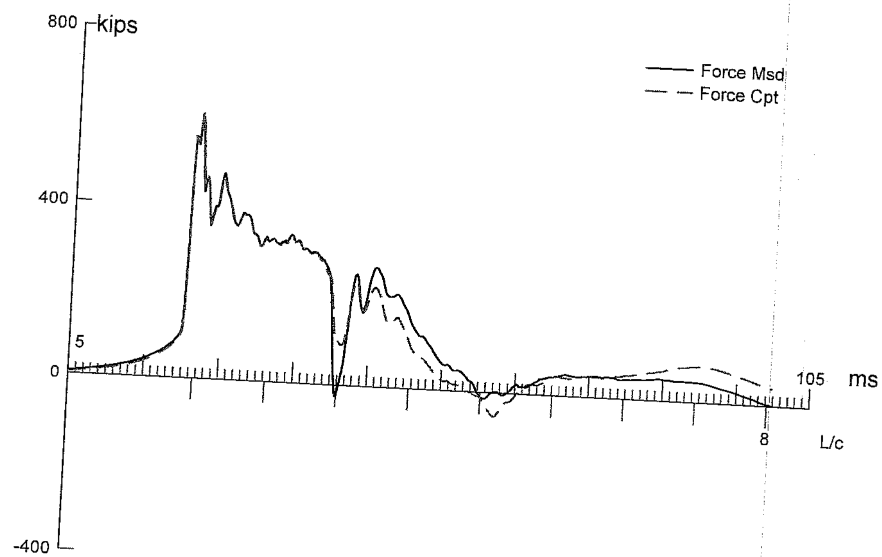
Toe Area 1.767 ft^2

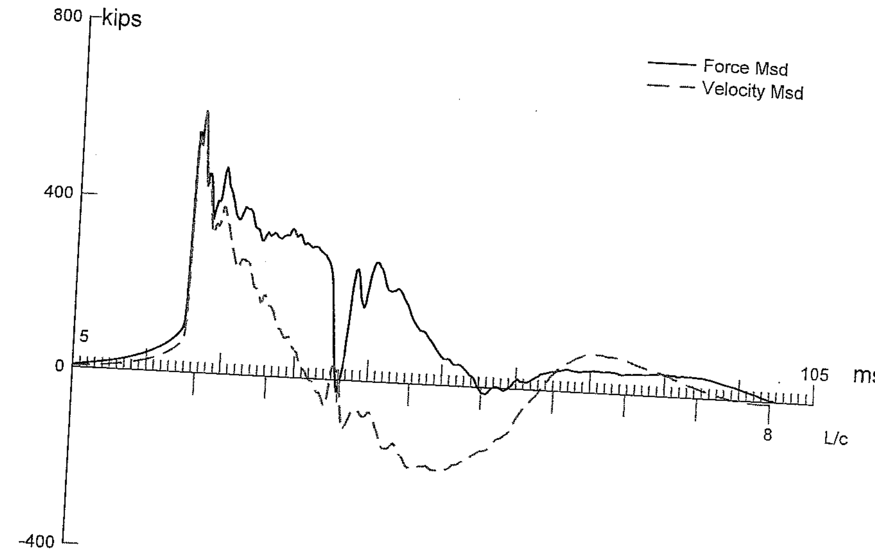
Top Segment Length 3.29 ft, Top Impedance 37.06 kips/ft/s

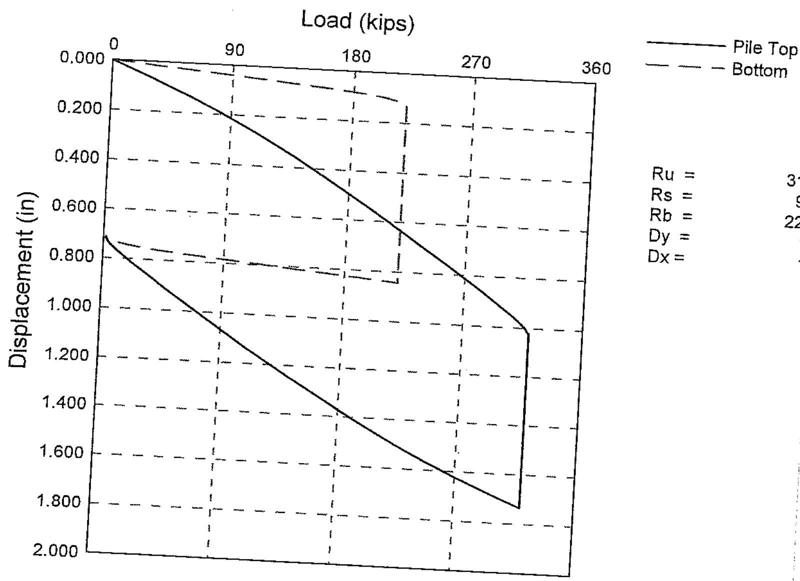
Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 16807.9 ft/s, 2L/c 16.4 ms

KIEWIT GENERAL; Pile: PILE 7 END DRIVE; PP18x0.375", D46-32; Blow: 236 (Test: 21-Apr-2010 15:05:)
Robert Miner Dynamic Testing, Inc.

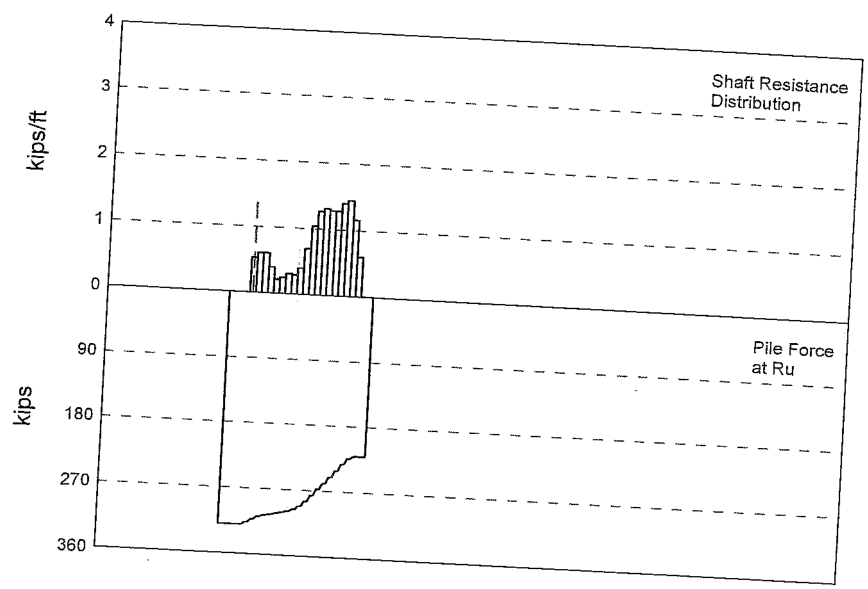
12-May-2010
CAPWAP(R) 2006-3







Ru = 319.8 kips
Rs = 99.8 kips
Rb = 220.0 kips
Dy = 1.02 in
Dx = 1.73 in



KIEWIT GENERAL; Pile: PILE 7 END DRIVE
PP18x0.375", D46-32; Blow: 236
Robert Miner Dynamic Testing, Inc.

Test: 21-Apr-2010 15:05:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 319.8; along Shaft 99.8; at Toe 220.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				319.8				
1	29.9	2.9	3.5	316.3	3.5	1.19	0.25	0.400
2	36.6	9.6	4.0	312.3	7.5	0.60	0.13	0.400
3	43.2	16.2	4.0	308.3	11.5	0.60	0.13	0.400
4	49.9	22.9	2.6	305.7	14.1	0.39	0.08	0.400
5	56.6	29.6	1.4	304.3	15.5	0.21	0.04	0.400
6	63.2	36.2	1.6	302.7	17.1	0.24	0.05	0.400
7	69.9	42.9	2.0	300.7	19.1	0.30	0.06	0.400
8	76.5	49.5	1.9	298.8	21.0	0.29	0.06	0.400
9	83.2	56.2	2.6	296.2	23.6	0.39	0.08	0.400
10	89.8	62.8	4.6	291.6	28.2	0.69	0.15	0.400
11	96.5	69.5	6.9	284.7	35.1	1.04	0.22	0.400
12	103.1	76.1	8.4	276.3	43.5	1.26	0.27	0.400
13	109.8	82.8	8.7	267.6	52.2	1.31	0.28	0.400
14	116.4	89.4	8.5	259.1	60.7	1.28	0.27	0.400
15	123.1	96.1	8.5	250.6	69.2	1.28	0.27	0.400
16	129.7	102.7	9.3	241.3	78.5	1.40	0.30	0.400
17	136.4	109.4	9.6	231.7	88.1	1.44	0.31	0.400
18	143.0	116.0	7.7	224.0	95.8	1.16	0.25	0.400
19	149.7	122.7	4.0	220.0	99.8	0.60	0.13	0.400
20	156.3	129.3	0.0	220.0	99.8	0.00	0.00	0.000
21	163.0	136.0	0.0	220.0	99.8	0.00	0.00	0.000
Avg. Shaft			4.8			0.73	0.16	0.400
Toe			220.0				1526.01	0.030

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.100	0.100
Case Damping Factor		1.077	0.178
Reloading Level	(% of Ru)	100	100
Soil Plug Weight	(kips)		0.05

max. Top Comp. Stress = 29.6 ksi (T= 22.0 ms, max= 1.042 x Top)
max. Comp. Stress = 30.9 ksi (Z= 29.9 ft, T= 23.6 ms)
max. Tens. Stress = -3.44 ksi (Z= 13.3 ft, T= 61.9 ms)
max. Energy (EMX) = 44.9 kip-ft; max. Measured Top Displ. (DMX)= 1.41 in

Analysis: 12-May-2010

KIEWIT GENERAL; Pile: PILE 7 END DRIVE
PP18x0.375", D46-32; Blow: 236
Robert Miner Dynamic Testing, Inc.

Test: 21-Apr-2010 15:05:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	615.5	-58.6	29.6	-2.82	44.89	15.8	1.410
2	6.7	616.4	-65.4	29.7	-3.15	44.63	15.8	1.390
5	16.6	619.6	-70.8	29.8	-3.41	44.01	15.7	1.339
8	26.6	633.7	-62.5	30.5	-3.01	43.56	15.2	1.294
11	36.6	628.9	-52.0	30.3	-2.50	41.53	14.7	1.243
14	46.6	585.0	-37.9	28.2	-1.83	37.74	14.3	1.189
17	56.6	577.3	-41.6	27.8	-2.00	36.25	14.0	1.140
20	66.5	567.1	-45.8	27.3	-2.21	34.68	13.7	1.091
23	76.5	567.6	-53.3	27.3	-2.57	33.46	13.3	1.039
26	86.5	558.5	-54.5	26.9	-2.62	31.64	12.8	0.991
29	96.5	563.4	-56.9	27.1	-2.74	29.88	11.9	0.942
32	106.4	505.6	-38.8	24.3	-1.87	25.67	11.1	0.894
35	116.4	491.7	-35.0	23.7	-1.69	23.30	10.2	0.849
38	126.4	434.6	-21.1	20.9	-1.01	19.43	9.4	0.805
41	136.4	420.1	-15.2	20.2	-0.73	17.29	8.6	0.763
44	146.4	356.8	0.0	17.2	0.00	13.90	8.6	0.723
45	149.7	357.8	-1.1	17.2	-0.05	13.78	8.8	0.710
46	153.0	316.0	0.0	15.2	0.00	13.00	9.9	0.697
47	156.3	301.5	-0.2	14.5	-0.01	12.88	11.0	0.683
48	159.7	290.4	-1.9	14.0	-0.09	12.76	10.4	0.669
49	163.0	289.9	-2.2	14.0	-0.11	12.88	10.0	0.655
Absolute	29.9			30.9			(T =	23.6 ms)
	13.3				-3.44		(T =	61.9 ms)

Analysis: 12-May-2010

Test: 21-Apr-2010 15:05:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

KIEWIT GENERAL; Pile: PILE 7 END DRIVE
PP18x0.375", D46-32; Blow: 236
Robert Miner Dynamic Testing, Inc.

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	586.8	533.9	481.1	428.2	375.3	322.4	269.5	216.6	163.8	110.9
RX	592.3	533.9	496.9	475.5	454.1	433.0	418.3	403.6	389.3	375.9
RU	609.8	559.2	508.6	458.0	407.4	356.8	306.2	255.7	205.1	154.5

RAU = 163.5 (kips); RA2 = 482.7 (kips)

Current CAPWAP Ru = 319.8 (kips); Corresponding J(RP)= 0.50;

RMX requires higher damping; see PDA-W

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips
16.46	21.77	552.6	563.1	607.8	1.407	0.689	0.714	45.1	510.9

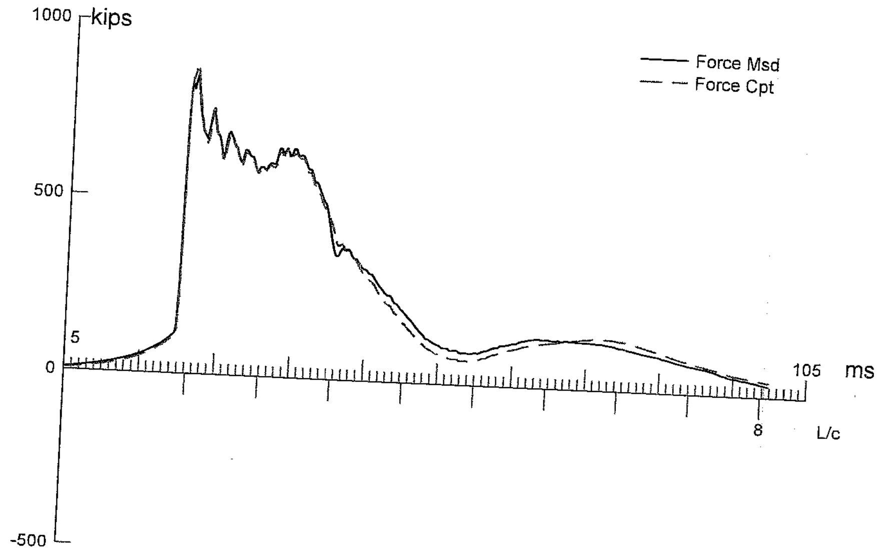
PILE PROFILE AND PILE MODEL

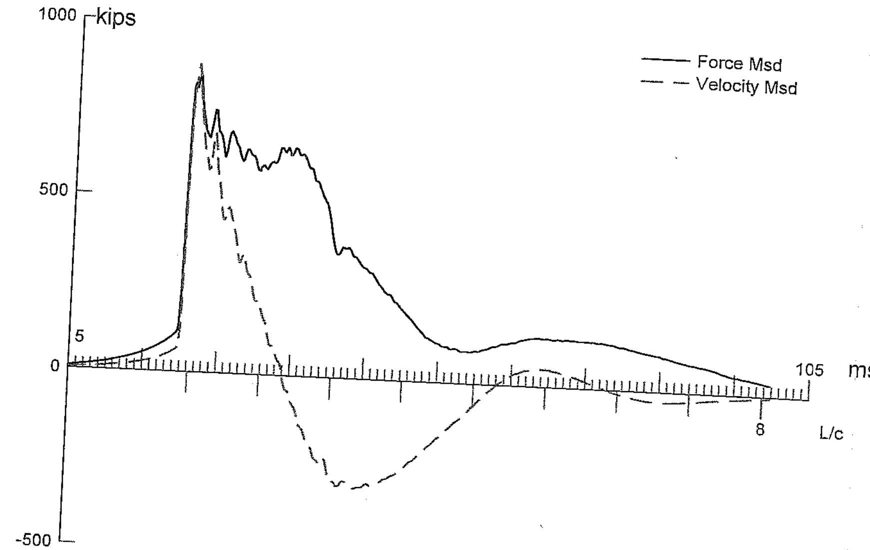
Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	20.76	29992.2	492.000	4.712
163.00	20.76	29992.2	492.000	4.712

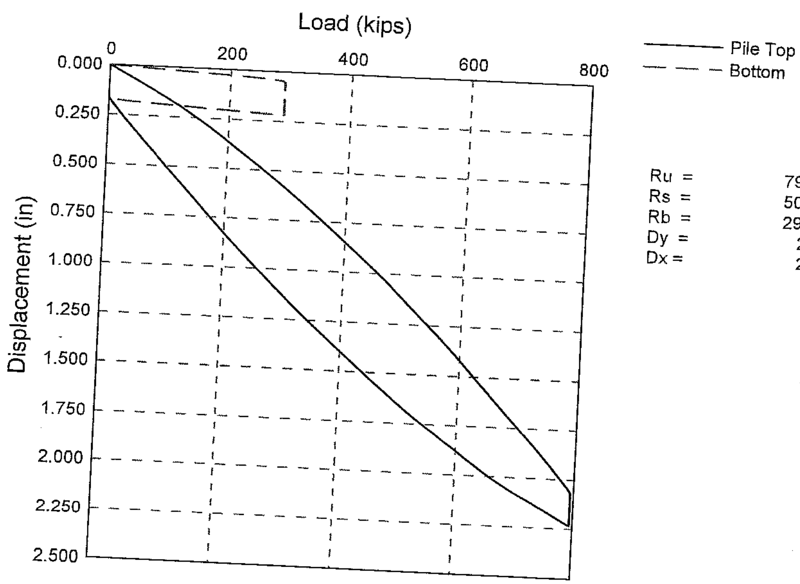
Toe Area 0.144 ft^2

Top Segment Length 3.33 ft, Top Impedance 37.06 kips/ft/s

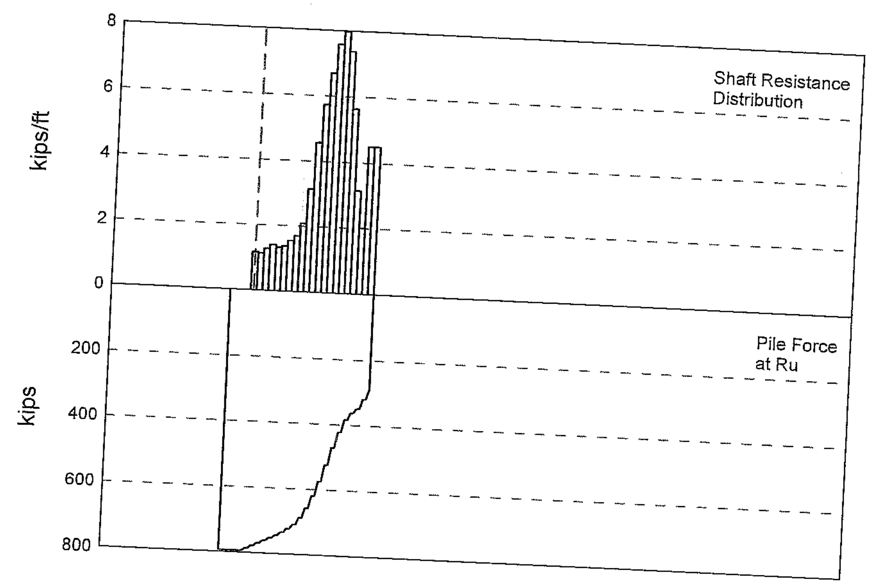
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 16807.9 ft/s, 2L/c 19.4 ms







Ru =	794.4 kips
Rs =	504.4 kips
Rb =	290.0 kips
Dy =	2.07 in
Dx =	2.23 in



GCC; Pile: PILE 7 1ST RESTRKE
PP18x0.375, D62-22; Blow: 3
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 08:23:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 794.4; along Shaft 504.4; at Toe 290.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft	Quake in
				794.4					
1	29.9	2.9	7.6	786.8	7.6	2.59	0.55	0.240	0.100
2	36.6	9.6	7.4	779.4	15.0	1.11	0.24	0.240	0.100
3	43.2	16.2	8.4	771.0	23.4	1.26	0.27	0.240	0.100
4	49.9	22.9	9.2	761.8	32.6	1.38	0.29	0.240	0.100
5	56.6	29.6	8.7	753.1	41.3	1.31	0.28	0.240	0.100
6	63.2	36.2	9.0	744.1	50.3	1.35	0.29	0.240	0.100
7	69.9	42.9	10.2	733.9	60.5	1.53	0.33	0.240	0.100
8	76.5	49.5	11.2	722.7	71.7	1.68	0.36	0.240	0.100
9	83.2	56.2	13.9	708.8	85.6	2.09	0.44	0.240	0.100
10	89.8	62.8	21.0	687.8	106.6	3.16	0.67	0.240	0.100
11	96.5	69.5	30.4	657.4	137.0	4.57	0.97	0.240	0.100
12	103.1	76.1	38.2	619.2	175.2	5.74	1.22	0.240	0.100
13	109.8	82.8	44.5	574.7	219.7	6.69	1.42	0.240	0.100
14	116.4	89.4	50.5	524.2	270.2	7.59	1.61	0.240	0.100
15	123.1	96.1	53.2	471.0	323.4	8.00	1.70	0.240	0.100
16	129.7	102.7	49.1	421.9	372.5	7.38	1.57	0.240	0.080
17	136.4	109.4	37.5	384.4	410.0	5.64	1.20	0.240	0.070
18	143.0	116.0	21.0	363.4	431.0	3.16	0.67	0.240	0.060
19	149.7	122.7	13.4	350.0	444.4	2.01	0.43	0.240	0.050
20	156.3	129.3	30.0	320.0	474.4	4.51	0.96	0.240	0.040
21	163.0	136.0	30.0	290.0	504.4	4.51	0.96	0.240	0.040
Avg. Shaft			24.0			3.71	0.79	0.240	0.090
Toe			290.0				2011.56	0.230	0.040

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		3.266	1.800
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	20	
Soil Plug Weight	(kips)		0.25

max. Top Comp. Stress = 42.4 ksi (T= 21.6 ms, max= 1.050 x Top)
max. Comp. Stress = 44.5 ksi (Z= 29.9 ft, T= 23.2 ms)
max. Tens. Stress = 0.00 ksi (Z= 3.3 ft, T= 0.0 ms)
max. Energy (EMX) = 102.2 kip-ft; max. Measured Top Displ. (DMX)= 1.89 in

GCC; Pile: PILE 7 1ST RESTRKE
PP18x0.375, D62-22; Blow: 3
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 08:23:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	880.3	0.0	42.4	0.00	102.17	22.4	1.892
2	6.7	881.8	0.0	42.5	0.00	100.99	22.3	1.849
5	16.6	887.1	0.0	42.7	0.00	97.27	22.1	1.718
8	26.6	911.5	0.0	43.9	0.00	93.30	21.4	1.582
11	36.6	900.5	0.0	43.4	0.00	85.41	20.4	1.445
14	46.6	843.7	0.0	40.6	0.00	74.38	19.5	1.309
17	56.6	829.4	0.0	39.9	0.00	66.89	18.5	1.175
20	66.5	771.9	0.0	37.2	0.00	57.29	17.5	1.043
23	76.5	789.5	0.0	38.0	0.00	50.67	16.3	0.911
26	86.5	765.1	0.0	36.8	0.00	41.45	14.7	0.779
29	96.5	764.4	0.0	36.8	0.00	34.42	12.3	0.652
32	106.4	634.2	0.0	30.5	0.00	23.25	9.9	0.534
35	116.4	584.2	0.0	28.1	0.00	17.00	7.7	0.424
38	126.4	479.6	0.0	23.1	0.00	9.26	5.9	0.327
41	136.4	441.4	0.0	21.3	0.00	5.91	4.8	0.242
44	146.4	386.6	0.0	18.6	0.00	3.21	4.3	0.167
45	149.7	388.7	0.0	18.7	0.00	2.85	4.1	0.143
46	153.0	373.1	0.0	18.0	0.00	2.32	3.9	0.120
47	156.3	373.9	0.0	18.0	0.00	1.99	3.5	0.098
48	159.7	346.1	0.0	16.7	0.00	1.46	2.7	0.077
49	163.0	347.0	0.0	16.7	0.00	1.13	2.2	0.056
Absolute	29.9			44.5			(T =	23.2 ms)
	3.3				0.00		(T =	0.0 ms)

Analysis: 12-May-2010

GCC; Pile: PILE 7 1ST RESTRKE
PP18x0.375, D62-22; Blow: 3
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 08:23:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1154.8	1100.9	1047.1	993.2	939.4	885.6	831.7	777.9	724.0	670.2
RX	1179.3	1124.4	1069.6	1014.7	959.8	904.9	850.0	795.1	743.4	708.5
RU	1319.3	1282.0	1244.6	1207.2	1169.8	1132.4	1095.0	1057.7	1020.3	982.9

RAU = 154.6 (kips); RA2 = 819.6 (kips)

Current CAPWAP Ru = 794.4 (kips); Corresponding J(RP)= 0.67; J(RX) = 0.70

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
23.97	21.37	877.5	815.7	857.2	1.892	0.129	0.167	102.8	1198.2

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	20.76	29992.2	492.000	4.712
163.00	20.76	29992.2	492.000	4.712

Toe Area 0.144 ft^2

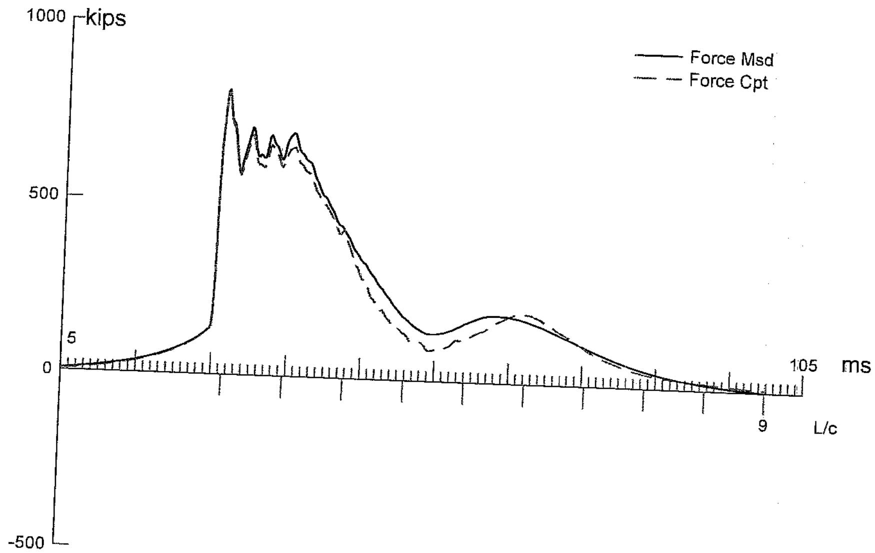
Top Segment Length 3.33 ft, Top Impedance 37.06 kips/ft/s

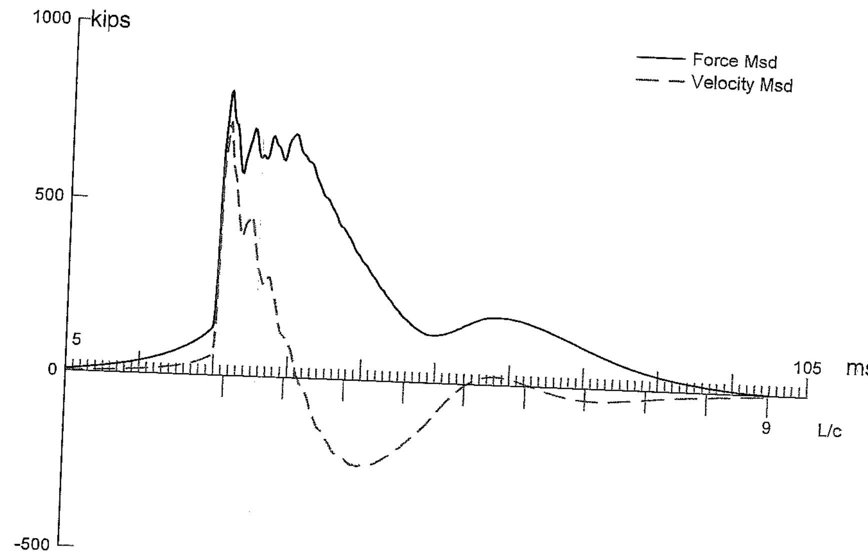
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 16807.9 ft/s, 2L/c 19.4 ms

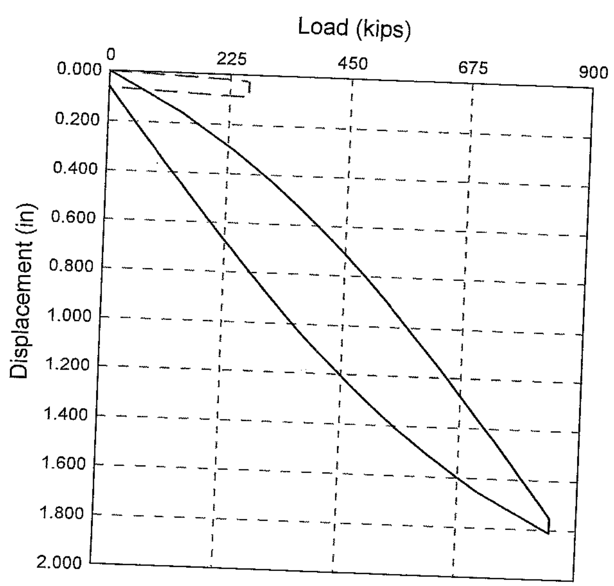
Analysis: 12-May-2010

GCC; Pile: PILE 7 2ND RESTRIKE; PP18x0.375, D62-22; Blow: 3 (Test: 03-May-2010 11:45:)
Robert Miner Dynamic Testing, Inc.

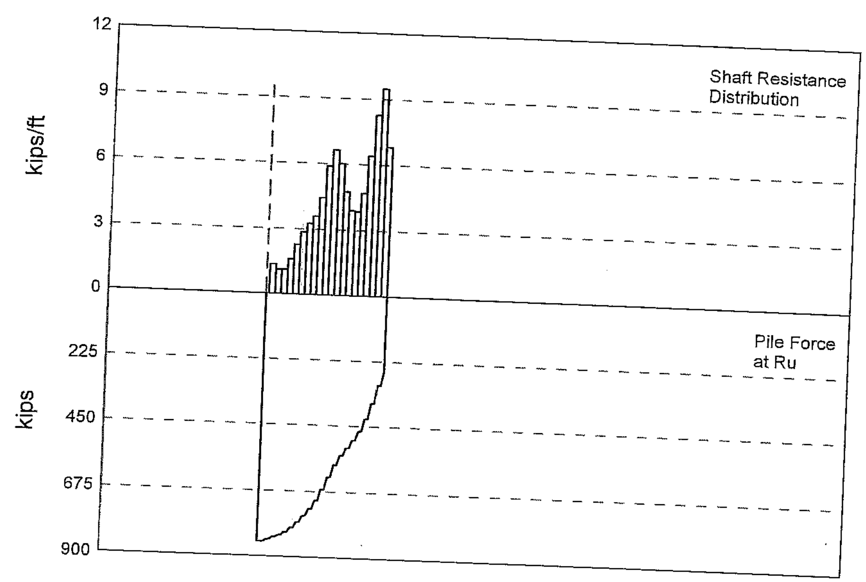
12-May-2010
CAPWAP(R) 2006-3







Ru = 850.1 kips
Rs = 590.1 kips
Rb = 260.0 kips
Dy = 1.75 in
Dx = 1.81 in



GCC; Pile: PILE 7 2ND RESTRIKE
PP18x0.375, D62-22; Blow: 3
Robert Miner Dynamic Testing, Inc.

Test: 03-May-2010 11:45:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 850.1; along Shaft 590.1; at Toe 260.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft	Quake in
				850.1					
1	10.0	9.0	8.8	841.3	8.8	0.98	0.21	0.160	0.100
2	16.7	15.7	7.3	834.0	16.1	1.09	0.23	0.160	0.100
3	23.4	22.4	7.3	826.7	23.4	1.09	0.23	0.160	0.100
4	30.1	29.1	10.5	816.2	33.9	1.57	0.33	0.160	0.100
5	36.8	35.8	15.1	801.1	49.0	2.26	0.48	0.160	0.100
6	43.4	42.4	19.2	781.9	68.2	2.87	0.61	0.160	0.100
7	50.1	49.1	21.8	760.1	90.0	3.26	0.69	0.160	0.100
8	56.8	55.8	24.1	736.0	114.1	3.61	0.77	0.160	0.100
9	63.5	62.5	29.8	706.2	143.9	4.46	0.95	0.160	0.100
10	70.2	69.2	39.3	666.9	183.2	5.88	1.25	0.160	0.100
11	76.9	75.9	44.4	622.5	227.6	6.64	1.41	0.160	0.100
12	83.5	82.5	40.2	582.3	267.8	6.02	1.28	0.160	0.100
13	90.2	89.2	31.8	550.5	299.6	4.76	1.01	0.160	0.100
14	96.9	95.9	26.1	524.4	325.7	3.91	0.83	0.160	0.100
15	103.6	102.6	25.8	498.6	351.5	3.86	0.82	0.160	0.080
16	110.3	109.3	31.5	467.1	383.0	4.71	1.00	0.160	0.060
17	117.0	116.0	42.7	424.4	425.7	6.39	1.36	0.160	0.040
18	123.6	122.6	55.4	369.0	481.1	8.29	1.76	0.160	0.030
19	130.3	129.3	63.3	305.7	544.4	9.47	2.01	0.160	0.025
20	137.0	136.0	45.7	260.0	590.1	6.84	1.45	0.160	0.022
Avg. Shaft			29.5			4.34	0.92	0.160	0.072
Toe			260.0				1803.46	0.037	0.022

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		2.548	0.260
Unloading Quake	(% of loading quake)	30	100
Reloading Level	(% of Ru)	100	100

max. Top Comp. Stress = 39.8 ksi (T= 26.8 ms, max= 1.018 x Top)
max. Comp. Stress = 40.5 ksi (Z= 10.0 ft, T= 27.2 ms)
max. Tens. Stress = 0.00 ksi (Z= 3.3 ft, T= 0.0 ms)
max. Energy (EMX) = 69.7 kip-ft; max. Measured Top Displ. (DMX)= 1.32 in

Analysis: 12-May-2010

GCC; Pile: PILE 7 2ND RESTRIKE
PP18x0.375, D62-22; Blow: 3
Robert Miner Dynamic Testing, Inc.

Test: 03-May-2010 11:45:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	826.5	0.0	39.8	0.00	69.72	19.2	1.374
2	6.7	832.7	0.0	40.1	0.00	68.77	19.0	1.335
4	13.4	812.1	0.0	39.1	0.00	64.36	18.6	1.258
6	20.0	798.8	0.0	38.5	0.00	60.51	18.2	1.180
8	26.7	790.2	0.0	38.0	0.00	56.79	17.7	1.102
10	33.4	776.2	0.0	37.4	0.00	52.52	17.0	1.023
12	40.1	751.6	0.0	36.2	0.00	47.59	16.2	0.943
14	46.8	717.7	0.0	34.6	0.00	42.33	15.3	0.864
16	53.5	680.7	0.0	32.8	0.00	37.12	14.3	0.786
18	60.1	646.0	0.0	31.1	0.00	31.84	13.2	0.703
20	66.8	620.1	0.0	29.9	0.00	26.56	11.9	0.623
22	73.5	571.2	0.0	27.5	0.00	21.24	10.7	0.548
24	80.2	563.3	0.0	27.1	0.00	16.37	9.5	0.477
26	86.9	538.7	0.0	25.9	0.00	12.49	8.6	0.410
28	93.6	490.2	0.0	23.6	0.00	9.71	7.9	0.349
30	100.2	464.9	0.0	22.4	0.00	7.56	7.2	0.289
32	106.9	464.4	0.0	22.4	0.00	5.64	6.5	0.230
34	113.6	426.4	0.0	20.5	0.00	4.01	5.6	0.177
36	120.3	381.5	0.0	18.4	0.00	2.57	4.7	0.128
38	127.0	338.0	0.0	16.3	0.00	1.35	3.9	0.083
40	133.7	299.2	0.0	14.4	0.00	0.49	3.6	0.042
41	137.0	305.7	0.0	14.7	0.00	0.30	2.8	0.023
Absolute	10.0			40.5			(T =	27.2 ms)
	3.3				0.00		(T =	0.0 ms)

Analysis: 12-May-2010

GCC; Pile: PILE 7 2ND RESTRIKE
PP18x0.375, D62-22; Blow: 3
Robert Miner Dynamic Testing, Inc.

Test: 03-May-2010 11:45:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1076.6	1033.1	989.5	946.0	902.5	859.0	815.5	771.9	728.4	684.9
RX	1100.5	1054.1	1007.7	961.4	915.0	868.6	822.2	775.8	729.4	684.9
RU	1232.6	1204.6	1176.7	1148.8	1120.9	1093.0	1065.0	1037.1	1009.2	981.3

RAU = 170.9 (kips); RA2 = 791.0 (kips)

Current CAPWAP Ru = 850.1 (kips); Corresponding J(RP)= 0.52; J(RX) = 0.54

VMX ft/s	TVP ms	VT1*Z kips	FT1 kips	FMX kips	DMX in	DFN in	SET in	EMX kip-ft	QUS kips
20.04	26.64	710.2	801.6	821.7	1.321	0.053	0.060	68.1	1184.4

PILE PROFILE AND PILE MODEL

Depth ft	Area in²	E-Modulus ksi	Spec. Weight lb/ft³	Perim. ft
0.00	20.76	29992.2	492.000	4.712
137.00	20.76	29992.2	492.000	4.712

Toe Area 0.144 ft²

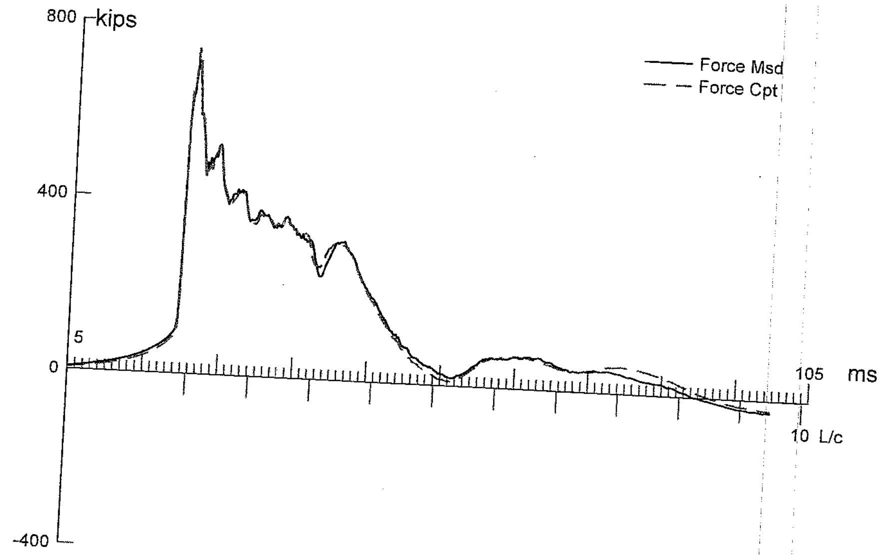
Top Segment Length 3.34 ft, Top Impedance 37.06 kips/ft/s

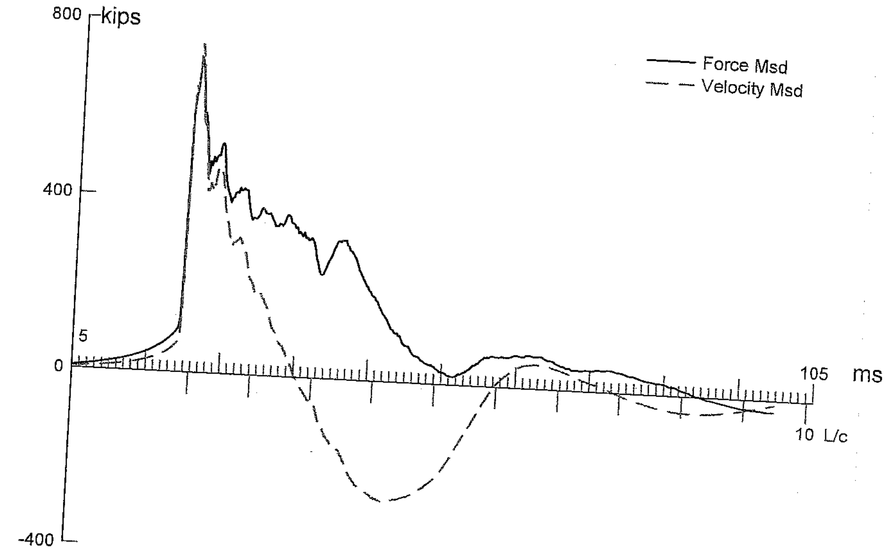
Pile Damping 1.0 %, Time Incr 0.199 ms, Wave Speed 16807.9 ft/s, 2L/c 16.3 ms

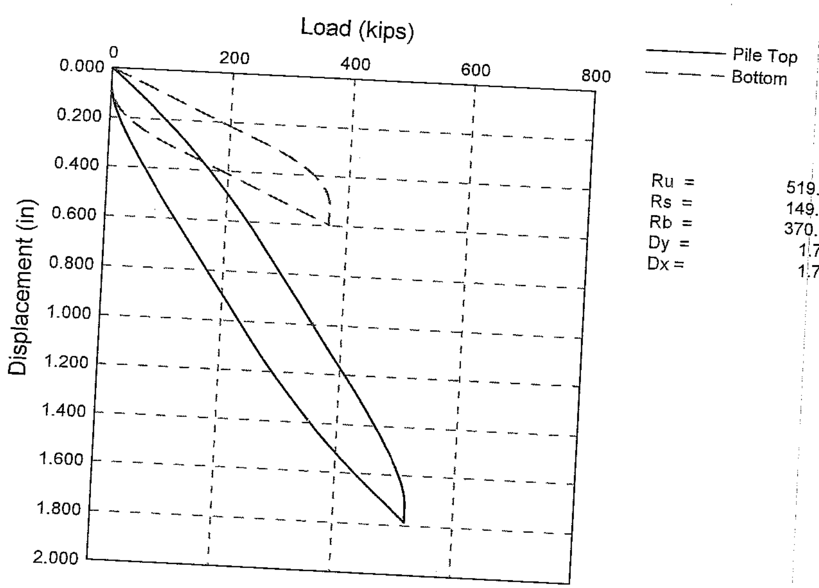
Analysis: 12-May-2010

KIEWIT GENERAL; Pile: PILE 8 END DRIVE; PP20x0.375", D46-32; Blow: 1052 (Test: 22-Apr-2010 11:11:)
Robert Miner Dynamic Testing, Inc.

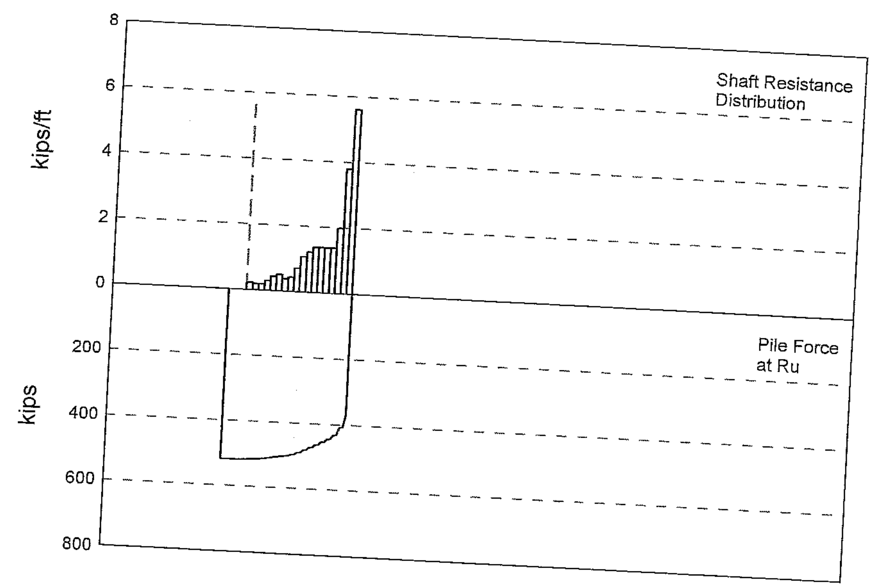
12-May-2010
CAPWAP(R) 2006-3







Ru = 519.9 kips
Rs = 149.9 kips
Rb = 370.0 kips
Dy = 1.71 in
Dx = 1.79 in



KIEWIT GENERAL; Pile: PILE 8 END DRIVE
PP20x0.375", D46-32; Blow: 1052
Robert Miner Dynamic Testing, Inc.

Test: 22-Apr-2010 11:11:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 519.9; along Shaft 149.9; at Toe 370.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				519.9				
1	26.7	6.7	1.5	518.4	1.5	0.23	0.04	0.400
2	33.3	13.3	1.3	517.1	2.8	0.20	0.04	0.400
3	40.0	20.0	1.3	515.8	4.1	0.20	0.04	0.400
4	46.7	26.7	2.0	513.8	6.1	0.30	0.06	0.400
5	53.3	33.3	2.9	510.9	9.0	0.44	0.08	0.400
6	60.0	40.0	3.3	507.6	12.3	0.50	0.09	0.400
7	66.7	46.7	2.5	505.1	14.8	0.38	0.07	0.400
8	73.3	53.3	2.9	502.2	17.7	0.44	0.08	0.400
9	80.0	60.0	4.7	497.5	22.4	0.71	0.13	0.400
10	86.7	66.7	7.1	490.4	29.5	1.07	0.20	0.400
11	93.3	73.3	8.1	482.3	37.6	1.22	0.23	0.400
12	100.0	80.0	9.1	473.2	46.7	1.37	0.26	0.400
13	106.7	86.7	9.1	464.1	55.8	1.37	0.26	0.400
14	113.3	93.3	9.1	455.0	64.9	1.37	0.26	0.400
15	120.0	100.0	9.1	445.9	74.0	1.37	0.26	0.400
16	126.7	106.7	13.2	432.7	87.2	1.98	0.38	0.400
17	133.3	113.3	25.3	407.4	112.5	3.80	0.72	0.400
18	140.0	120.0	37.4	370.0	149.9	5.61	1.07	0.400
Avg. Shaft			8.3			1.25	0.24	0.400
Toe			370.0				169.60	0.045

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.100	0.400
Case Damping Factor		1.453	0.403
Damping Type			Smith
Unloading Quake	(% of loading quake)	70	100
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	70	
Soil Plug Weight	(kips)		0.05

max. Top Comp. Stress = 32.1 ksi (T= 21.2 ms, max= 1.011 x Top)
max. Comp. Stress = 32.5 ksi (Z= 26.7 ft, T= 22.6 ms)
max. Tens. Stress = -1.99 ksi (Z= 80.0 ft, T= 60.9 ms)
max. Energy (EMX) = 54.0 kip-ft; max. Measured Top Displ. (DMX)= 1.42 in

KIEWIT GENERAL; Pile: PILE 8 END DRIVE
PP20x0.375", D46-32; Blow: 1052
Robert Miner Dynamic Testing, Inc.

Test: 22-Apr-2010 11:11:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	743.1	-23.8	32.1	-1.03	54.05	17.4	1.435
2	6.7	743.1	-25.0	32.1	-1.08	53.75	17.4	1.414
4	13.3	743.2	-27.1	32.1	-1.17	53.10	17.3	1.372
6	20.0	746.6	-28.4	32.3	-1.23	52.59	17.1	1.335
8	26.7	751.1	-29.6	32.5	-1.28	52.03	16.9	1.296
10	33.3	743.9	-30.0	32.2	-1.30	50.65	16.7	1.254
12	40.0	740.0	-32.0	32.0	-1.39	49.28	16.5	1.209
14	46.7	738.6	-37.6	31.9	-1.62	47.84	16.2	1.160
16	53.3	733.8	-41.9	31.7	-1.81	46.11	15.9	1.110
18	60.0	723.3	-42.9	31.3	-1.85	44.03	15.5	1.060
20	66.7	710.9	-42.9	30.7	-1.86	41.83	15.2	1.008
22	73.3	708.7	-44.6	30.6	-1.93	39.93	14.8	0.955
24	80.0	709.8	-46.0	30.7	-1.99	37.95	14.3	0.901
26	86.7	701.7	-44.8	30.3	-1.94	35.48	13.7	0.847
28	93.3	680.7	-40.0	29.4	-1.73	32.43	13.1	0.791
30	100.0	654.9	-34.2	28.3	-1.48	29.30	12.5	0.736
32	106.7	624.3	-27.5	27.0	-1.19	26.17	11.9	0.680
34	113.3	595.4	-21.3	25.7	-0.92	23.29	11.3	0.626
36	120.0	573.4	-15.7	24.8	-0.68	20.65	10.6	0.574
38	126.7	566.9	-10.0	24.5	-0.43	18.25	9.6	0.522
40	133.3	497.4	-0.4	21.5	-0.02	15.52	10.8	0.471
42	140.0	452.0	0.0	19.5	0.00	8.15	9.9	0.423
Absolute	26.7			32.5			(T =	22.6 ms)
	80.0				-1.99		(T =	60.9 ms)

Analysis: 12-May-2010

KIEWIT GENERAL; Pile: PILE 8 END DRIVE
PP20x0.375", D46-32; Blow: 1052
Robert Miner Dynamic Testing, Inc.

Test: 22-Apr-2010 11:11:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	839.0	789.1	739.3	689.5	639.7	589.8	540.0	490.2	440.4	390.5
RX	901.6	843.3	785.0	726.7	668.4	610.2	582.6	557.3	532.0	506.7
RU	861.0	813.4	765.8	718.1	670.5	622.9	575.3	527.6	480.0	432.4

RAU = 285.8 (kips); RA2 = 582.9 (kips)

Current CAPWAP Ru = 519.9 (kips); Corresponding J(RP)= 0.64; J(RX) = 0.85

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
18.49	21.02	663.8	673.4	728.3	1.425	0.066	0.077	54.3	868.0

PILE PROFILE AND PILE MODEL

Depth ft	Area in^2	E-Modulus ksi	Spec. Weight lb/ft^3	Perim. ft
0.00	23.12	29992.2	492.000	5.236
140.00	23.12	29992.2	492.000	5.236

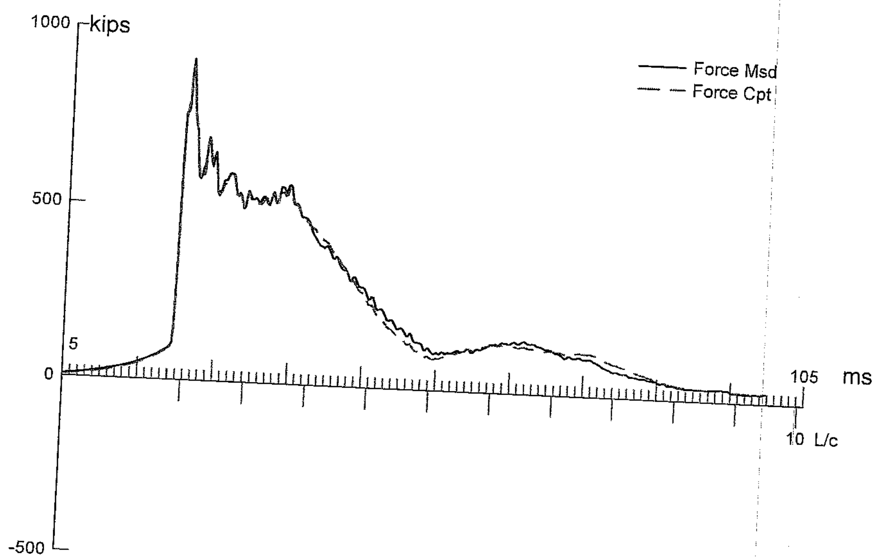
Toe Area 2.182 ft^2

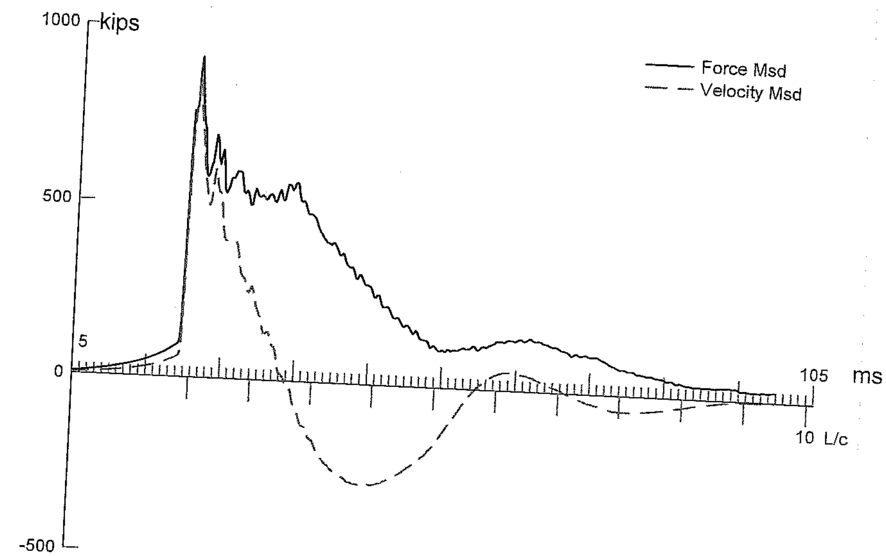
Top Segment Length 3.33 ft, Top Impedance 41.27 kips/ft/s

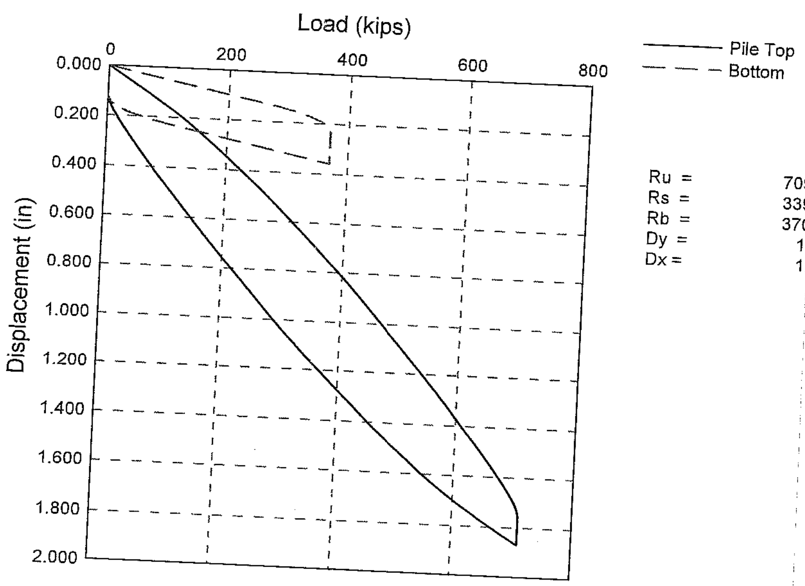
Pile Damping 2.0 %, Time Incr 0.198 ms, Wave Speed 16807.9 ft/s, 2L/c 16.7 ms

GCC; Pile: P8 START 1ST RESTRKE; PP20x0.375, D62-22; Blow: 7 (Test: 26-Apr-2010 09:09:)
Robert Miner Dynamic Testing, Inc.

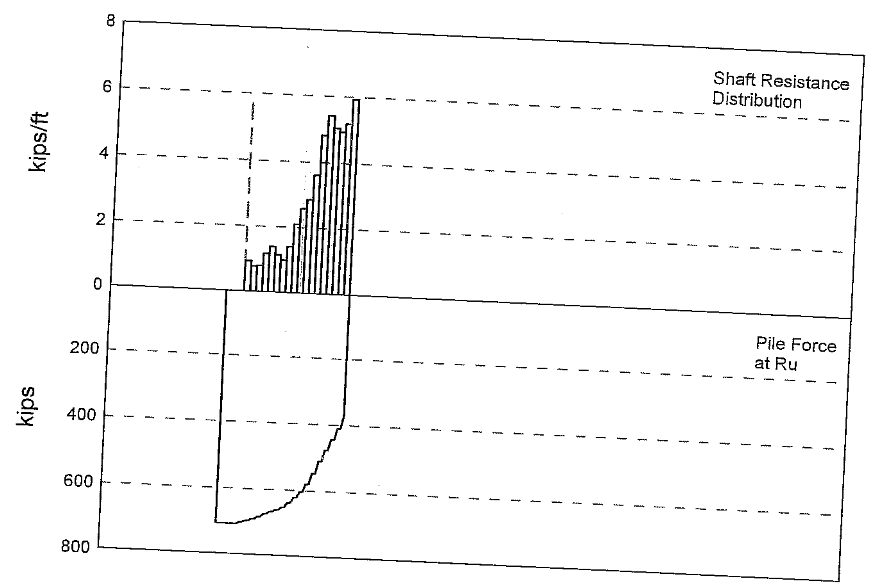
12-May-2010
CAPWAP(R) 2006-3







Ru = 709.9 kips
Rs = 339.9 kips
Rb = 370.0 kips
Dy = 1.75 in
Dx = 1.87 in



Test: 26-Apr-2010 09:09:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

GCC; Pile: P8 START 1ST RESTRKE
PP20x0.375, D62-22; Blow: 7
Robert Miner Dynamic Testing, Inc.

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 709.9; along Shaft 339.9; at Toe 370.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				709.9				
1	26.7	6.7	6.1	703.8	6.1	0.92	0.17	0.260
2	33.3	13.3	5.0	698.8	11.1	0.75	0.14	0.260
3	40.0	20.0	5.2	693.6	16.3	0.78	0.15	0.260
4	46.7	26.7	7.7	685.9	24.0	1.16	0.22	0.260
5	53.3	33.3	9.1	676.8	33.1	1.37	0.26	0.260
6	60.0	40.0	7.5	669.3	40.6	1.13	0.21	0.260
7	66.7	46.7	6.5	662.8	47.1	0.98	0.19	0.260
8	73.3	53.3	9.2	653.6	56.3	1.38	0.26	0.260
9	80.0	60.0	14.0	639.6	70.3	2.10	0.40	0.260
10	86.7	66.7	17.2	622.4	87.5	2.58	0.49	0.260
11	93.3	73.3	19.1	603.3	106.6	2.87	0.55	0.260
12	100.0	80.0	24.1	579.2	130.7	3.62	0.69	0.260
13	106.7	86.7	32.2	547.0	162.9	4.83	0.92	0.260
14	113.3	93.3	36.2	510.8	199.1	5.43	1.04	0.260
15	120.0	100.0	33.7	477.1	232.8	5.06	0.97	0.260
16	126.7	106.7	32.9	444.2	265.7	4.94	0.94	0.260
17	133.3	113.3	34.6	409.6	300.3	5.19	0.99	0.260
18	140.0	120.0	39.6	370.0	339.9	5.94	1.13	0.260
Avg. Shaft			18.9			2.83	0.54	0.260
Toe			370.0				169.60	0.070

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.100	0.180
Case Damping Factor		2.142	0.628
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	15	

max. Top Comp. Stress = 38.8 ksi (T= 21.4 ms, max= 1.039 x Top)
max. Comp. Stress = 40.4 ksi (Z= 26.7 ft, T= 22.8 ms)
max. Tens. Stress = 0.00 ksi (Z= 3.3 ft, T= 0.0 ms)
max. Energy (EMX) = 78.1 kip-ft; max. Measured Top Displ. (DMX)= 1.57 in

GCC; Pile: P8 START 1ST RESTRKE
PP20x0.375, D62-22; Blow: 7
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 09:09:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	898.2	0.0	38.8	0.00	78.08	20.7	1.573
2	6.7	899.6	0.0	38.9	0.00	77.31	20.6	1.541
4	13.3	902.8	0.0	39.0	0.00	75.75	20.5	1.475
6	20.0	917.9	0.0	39.7	0.00	74.16	20.2	1.409
8	26.7	933.6	0.0	40.4	0.00	72.48	19.8	1.341
10	33.3	911.8	0.0	39.4	0.00	68.19	19.4	1.271
12	40.0	901.8	0.0	39.0	0.00	64.53	18.9	1.203
14	46.7	894.6	0.0	38.7	0.00	60.83	18.3	1.132
16	53.3	871.1	0.0	37.7	0.00	56.47	17.8	1.062
18	60.0	838.4	0.0	36.3	0.00	51.80	17.3	0.989
20	66.7	819.3	0.0	35.4	0.00	47.84	16.8	0.917
22	73.3	814.7	0.0	35.2	0.00	44.28	16.0	0.844
24	80.0	803.6	0.0	34.7	0.00	40.29	15.1	0.771
26	86.7	773.8	0.0	33.5	0.00	35.70	14.2	0.699
28	93.3	737.9	0.0	31.9	0.00	31.02	13.1	0.628
30	100.0	706.4	0.0	30.5	0.00	26.56	11.8	0.558
32	106.7	664.6	0.0	28.7	0.00	22.09	10.5	0.490
34	113.3	597.8	0.0	25.9	0.00	17.61	9.3	0.427
36	120.0	557.5	0.0	24.1	0.00	13.63	8.2	0.368
38	126.7	529.0	0.0	22.9	0.00	10.52	7.2	0.313
40	133.3	490.9	0.0	21.2	0.00	8.05	6.8	0.262
42	140.0	449.5	0.0	19.4	0.00	5.21	5.6	0.214
Absolute	26.7			40.4			(T =	22.8 ms)
	3.3				0.00		(T =	0.0 ms)

GCC; Pile: P8 START 1ST RESTRKE
PP20x0.375, D62-22; Blow: 7
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 09:09:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1171.8	1115.6	1059.3	1003.0	946.7	890.5	834.2	777.9	721.7	665.4
RX	1203.5	1144.0	1084.5	1024.9	965.4	905.9	846.3	786.8	727.3	667.7
RU	1244.9	1195.9	1146.9	1098.0	1049.0	1000.1	951.1	902.1	853.2	804.2

RAU = 252.4 (kips); RA2 = 781.9 (kips)

Current CAPWAP Ru = 709.9 (kips); Corresponding J(RP)= 0.82; J(RX) = 0.83

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
21.20	21.02	868.5	866.0	938.0	1.575	0.114	0.125	78.6	1109.6

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	23.12	29992.2	492.000	5.236
140.00	23.12	29992.2	492.000	5.236

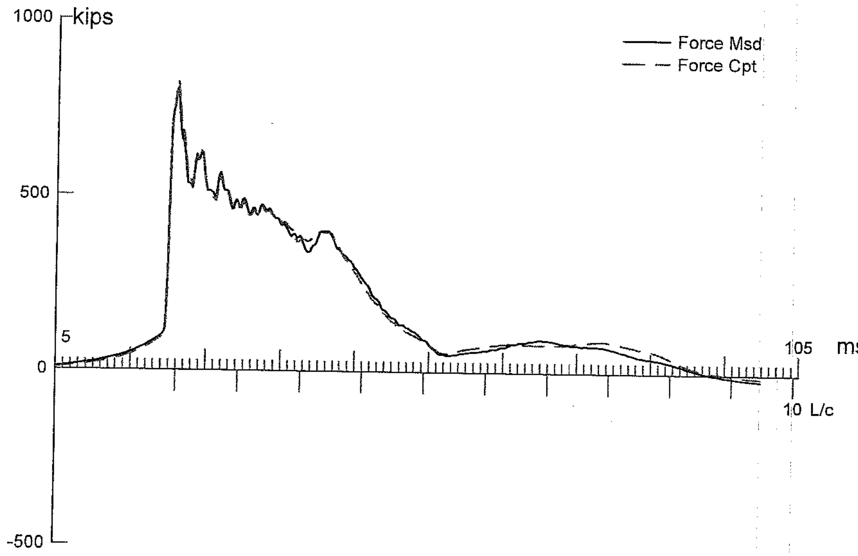
Toe Area 2.182 ft^2

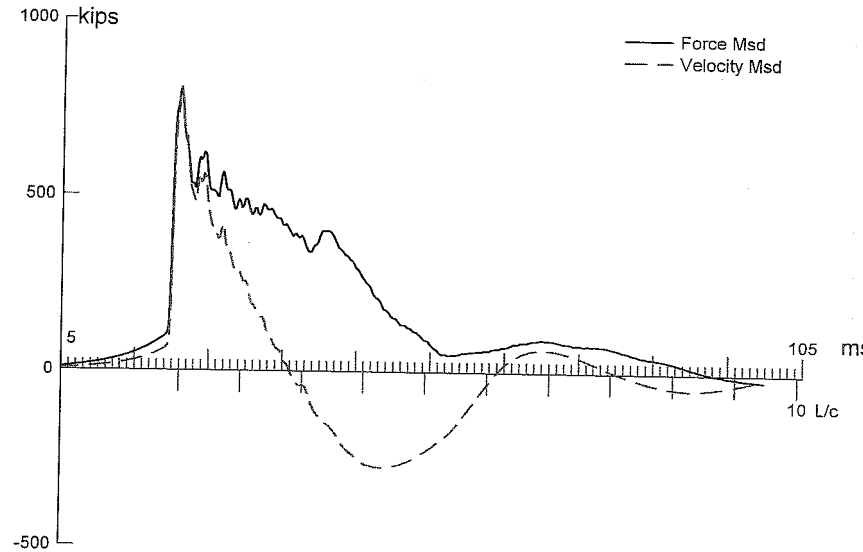
Top Segment Length 3.33 ft, Top Impedance 41.27 kips/ft/s

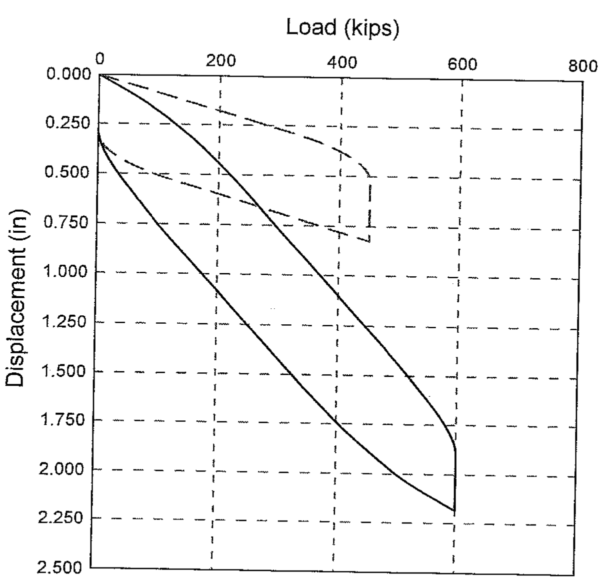
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 16807.9 ft/s, 2L/c 16.7 ms

GCC; Pile: PILE 8 END 1ST RESTRKE; PP20x0.375, D62-22; Blow: 286 (Test: 26-Apr-2010 09:20:)
Robert Miner Dynamic Testing, Inc.

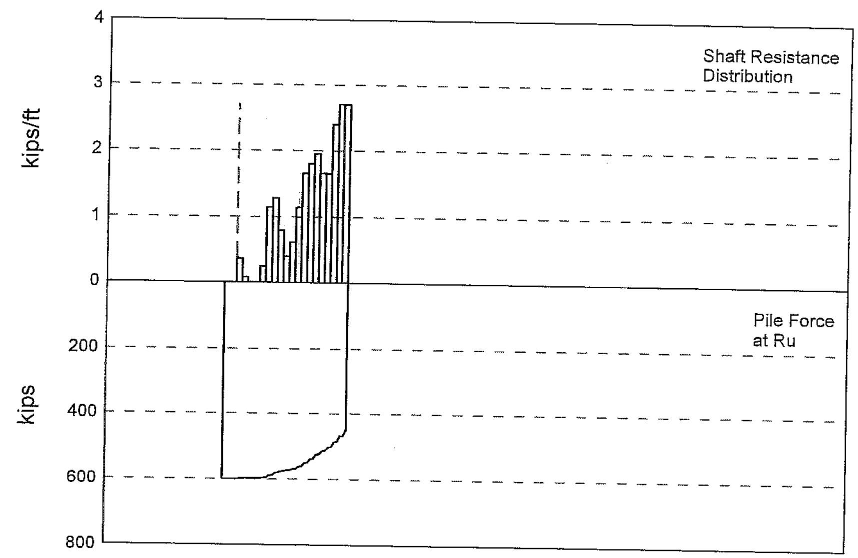
12-May-2010
CAPWAP(R) 2006-3







Ru =	600.2 kips
Rs =	150.2 kips
Rb =	450.0 kips
Dy =	1.89 in
Dx =	2.19 in



GCC; Pile: PILE 8 END 1ST RESTRKE
PP20x0.375, D62-22; Blow: 286
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 09:20:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 600.2; along Shaft 150.2; at Toe 450.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft
				600.2				
1	20.0	6.0	2.4	597.8	2.4	0.40	0.08	0.330
2	26.7	12.7	0.5	597.3	2.9	0.08	0.01	0.330
3	33.3	19.3	0.0	597.3	2.9	0.00	0.00	0.000
4	40.0	26.0	0.0	597.3	2.9	0.00	0.00	0.000
5	46.7	32.7	1.6	595.7	4.5	0.24	0.05	0.330
6	53.3	39.3	7.6	588.1	12.1	1.14	0.22	0.330
7	60.0	46.0	8.5	579.6	20.6	1.28	0.24	0.330
8	66.7	52.7	5.3	574.3	25.9	0.80	0.15	0.330
9	73.3	59.3	2.6	571.7	28.5	0.39	0.07	0.330
10	80.0	66.0	4.1	567.6	32.6	0.62	0.12	0.330
11	86.7	72.7	7.6	560.0	40.2	1.14	0.22	0.330
12	93.3	79.3	11.0	549.0	51.2	1.65	0.32	0.330
13	100.0	86.0	12.0	537.0	63.2	1.80	0.34	0.330
14	106.7	92.7	13.0	524.0	76.2	1.95	0.37	0.330
15	113.3	99.3	11.0	513.0	87.2	1.65	0.32	0.330
16	120.0	106.0	11.0	502.0	98.2	1.65	0.32	0.330
17	126.7	112.7	16.0	486.0	114.2	2.40	0.46	0.330
18	133.3	119.3	18.0	468.0	132.2	2.70	0.52	0.330
19	140.0	126.0	18.0	450.0	150.2	2.70	0.52	0.330
Avg. Shaft			7.9			1.19	0.23	0.330
Toe			450.0				206.26	0.055

Soil Model Parameters/Extensions		Shaft	Toe
Quake	(in)	0.064	0.403
Case Damping Factor		1.201	0.600
Unloading Quake	(% of loading quake)	30	50
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	30	

max. Top Comp. Stress = 35.6 ksi (T= 21.4 ms, max= 1.045 x Top)
max. Comp. Stress = 37.1 ksi (Z= 53.3 ft, T= 24.4 ms)
max. Tens. Stress = -1.49 ksi (Z= 93.3 ft, T= 63.7 ms)
max. Energy (EMX) = 75.3 kip-ft; max. Measured Top Displ. (DMX)= 1.69 in

GCC; Pile: PILE 8 END 1ST RESTRKE
PP20x0.375, D62-22; Blow: 286
Robert Miner Dynamic Testing, Inc.

Test: 26-Apr-2010 09:20:
CAPWAP(R) 2006-3
OP: RMDT:--RMINER

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	822.3	-11.0	35.6	-0.48	75.29	19.0	1.686
2	6.7	823.4	-11.9	35.6	-0.52	74.82	19.0	1.660
4	13.3	830.8	-13.5	35.9	-0.59	74.01	18.8	1.613
6	20.0	835.9	-15.5	36.1	-0.67	73.15	18.7	1.564
8	26.7	821.9	-15.7	35.5	-0.68	70.84	18.6	1.513
10	33.3	821.1	-17.4	35.5	-0.75	69.50	18.5	1.459
12	40.0	827.4	-18.6	35.8	-0.81	68.30	18.3	1.401
14	46.7	846.4	-20.5	36.6	-0.89	67.02	17.8	1.341
16	53.3	859.0	-21.1	37.1	-0.91	64.89	17.3	1.279
18	60.0	826.5	-18.9	35.7	-0.82	60.22	16.8	1.217
20	66.7	782.6	-16.9	33.8	-0.73	55.39	16.5	1.154
22	73.3	760.9	-21.5	32.9	-0.93	51.95	16.2	1.091
24	80.0	763.9	-27.9	33.0	-1.21	49.49	15.7	1.026
26	86.7	765.3	-32.9	33.1	-1.42	46.57	15.0	0.959
28	93.3	749.5	-34.5	32.4	-1.49	42.80	14.3	0.894
30	100.0	716.9	-33.8	31.0	-1.46	38.44	13.6	0.830
32	106.7	678.4	-32.3	29.3	-1.40	34.20	12.9	0.768
34	113.3	635.6	-30.9	27.5	-1.33	30.12	12.3	0.707
36	120.0	610.2	-30.3	26.4	-1.31	26.79	11.5	0.648
38	126.7	590.9	-30.1	25.6	-1.30	23.77	10.7	0.591
40	133.3	573.4	-26.6	24.8	-1.15	20.35	11.1	0.534
42	140.0	540.9	-19.5	23.4	-0.84	15.69	10.2	0.480
Absolute	53.3			37.1			(T =	24.4 ms)
	93.3				-1.49		(T =	63.7 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1004.2	946.6	888.9	831.3	773.7	716.0	658.4	600.7	543.1	485.5
RX	1021.8	961.9	902.0	842.1	782.2	722.3	680.0	648.8	617.9	589.2
RU	1006.8	949.4	892.0	834.6	777.2	719.8	662.4	605.1	547.7	490.3

RAU = 314.4 (kips); RA2 = 712.6 (kips)

Current CAPWAP Ru = 600.2 (kips); Corresponding J(RP)= 0.70; J(RX) = 0.86

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
19.64	21.22	787.1	793.5	811.8	1.691	0.296	0.300	75.8	913.6

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	23.12	29992.2	492.000	5.236
140.00	23.12	29992.2	492.000	5.236

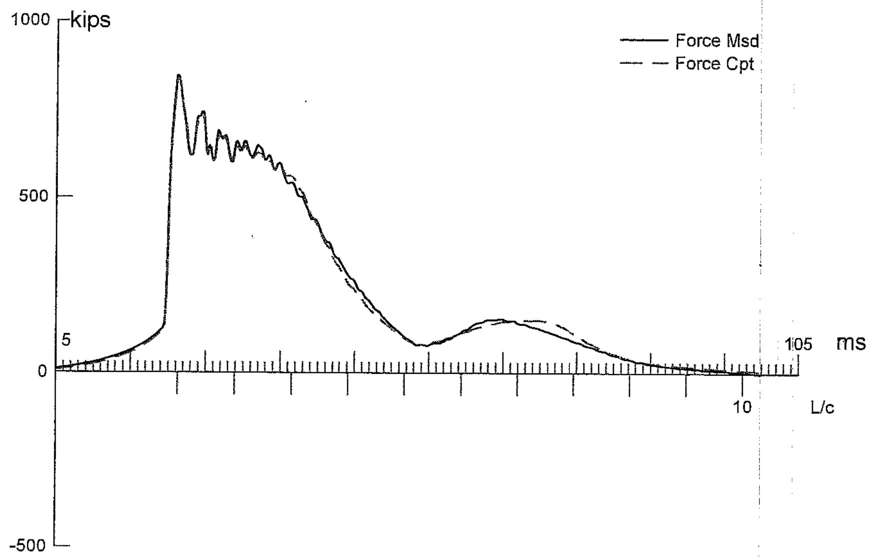
Toe Area 2.182 ft^2

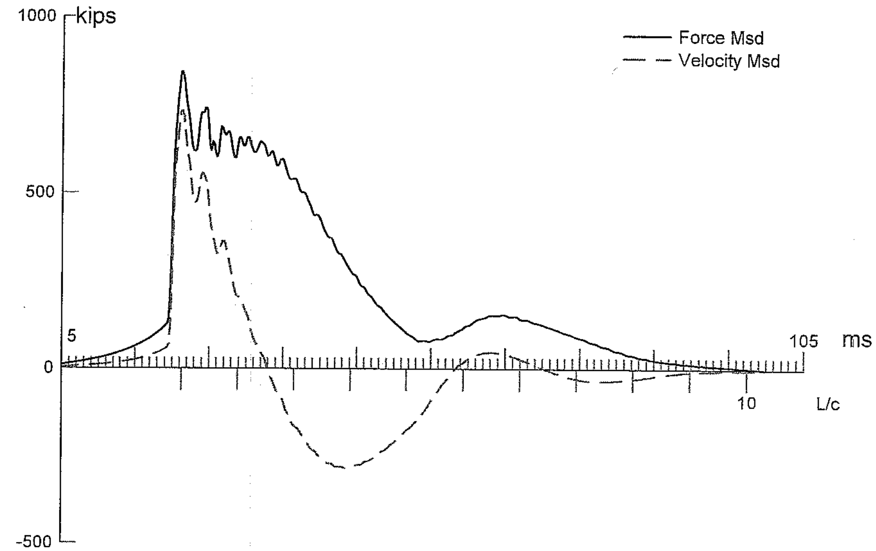
Top Segment Length 3.33 ft, Top Impedance 41.27 kips/ft/s

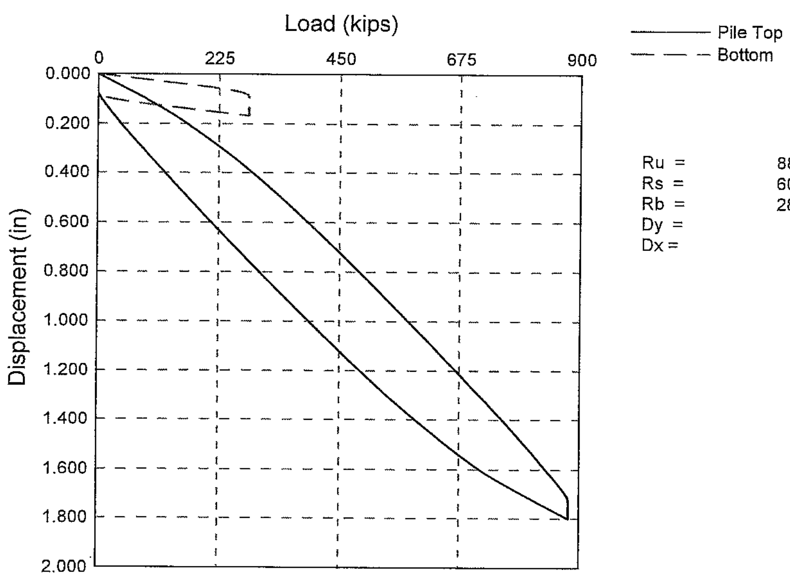
Pile Damping 1.0 %, Time Incr 0.198 ms, Wave Speed 16807.9 ft/s, 2L/c 16.7 ms

GCC; Pile: PILE 8 2ND RESTRIKE; PP20X0.375", D62-22; Blow: 5 (Test: 03-May-2010 10:18:)
Robert Miner Dynamic Testing, Inc.

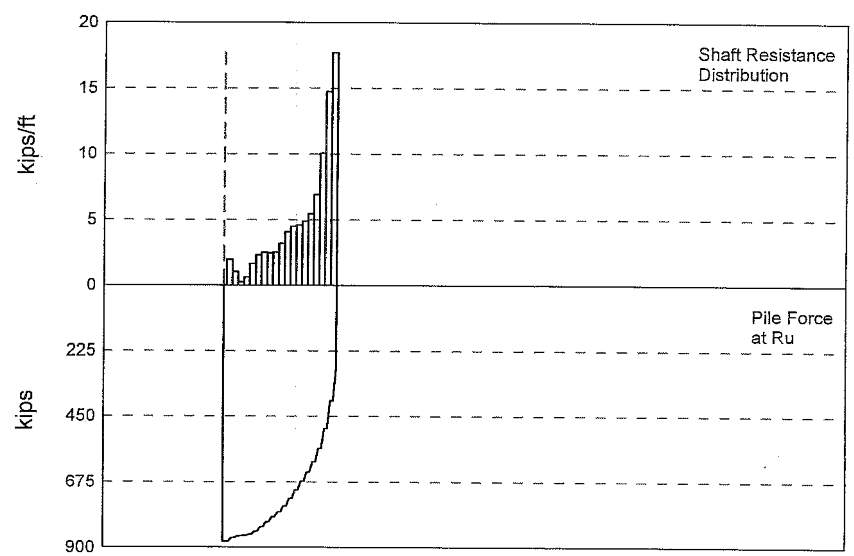
12-May-2010
CAPWAP(R) 2006-3







Ru =	880.2 kips
Rs =	600.2 kips
Rb =	280.0 kips
Dy =	1.72 in
Dx =	1.80 in



GCC; Pile: PILE 8 2ND RESTRIKE — Test: 03-May-2010 10:18:
PP20X0.375", D62-22; Blow: 5 — CAPWAP(R) 2006-3
Robert Miner Dynamic Testing, Inc. — OP: RMDT:--RMINER

CAPWAP SUMMARY RESULTS

Total CAPWAP Capacity: 880.2; along Shaft 600.2; at Toe 280.0 kips

Soil Sgmnt No.	Dist. Below Gages ft	Depth Below Grade ft	Ru kips	Force in Pile kips	Sum of Ru kips	Unit Resist. (Depth) kips/ft	Unit Resist. (Area) ksf	Smith Damping Factor s/ft	Quake in
				880.2					
1	9.8	7.8	12.8	867.4	12.8	1.63	0.31	0.200	0.100
2	16.4	14.4	6.7	860.7	19.5	1.02	0.19	0.200	0.100
3	23.0	21.0	1.7	859.0	21.2	0.26	0.05	0.200	0.100
4	29.5	27.5	4.1	854.9	25.3	0.62	0.12	0.200	0.100
5	36.1	34.1	10.8	844.1	36.1	1.65	0.31	0.200	0.100
6	42.7	40.7	15.2	828.9	51.3	2.32	0.44	0.200	0.100
7	49.2	47.2	16.4	812.5	67.7	2.50	0.48	0.200	0.100
8	55.8	53.8	16.2	796.3	83.9	2.47	0.47	0.200	0.100
9	62.4	60.4	16.6	779.7	100.5	2.53	0.48	0.200	0.100
10	68.9	66.9	20.9	758.8	121.4	3.18	0.61	0.200	0.100
11	75.5	73.5	26.8	732.0	148.2	4.08	0.78	0.200	0.100
12	82.1	80.1	29.6	702.4	177.8	4.51	0.86	0.200	0.100
13	88.6	86.6	30.2	672.2	208.0	4.60	0.88	0.200	0.100
14	95.2	93.2	32.1	640.1	240.1	4.89	0.93	0.200	0.100
15	101.7	99.7	35.8	604.3	275.9	5.45	1.04	0.200	0.100
16	108.3	106.3	45.2	559.1	321.1	6.89	1.32	0.200	0.100
17	114.9	112.9	66.1	493.0	387.2	10.07	1.92	0.200	0.100
18	121.4	119.4	96.9	396.1	484.1	14.76	2.82	0.200	0.081
19	128.0	126.0	116.1	280.0	600.2	17.69	3.38	0.200	0.030
Avg. Shaft			31.6			4.76	0.91	0.200	0.083
Toe			280.0				128.34	0.080	0.070

Soil Model Parameters/Extensions		Shaft	Toe
Case Damping Factor		2.909	0.543
Unloading Quake	(% of loading quake)	60	90
Reloading Level	(% of Ru)	100	100
Unloading Level	(% of Ru)	10	

max. Top Comp. Stress = 36.7 ksi (T= 21.7 ms, max= 1.018 x Top)
max. Comp. Stress = 37.4 ksi (Z= 9.8 ft, T= 22.1 ms)
max. Tens. Stress = 0.00 ksi (Z= 3.3 ft, T= 0.0 ms)
max. Energy (EMX) = 76.9 kip-ft; max. Measured Top Displ. (DMX)= 1.42 in

EXTREMA TABLE

Pile Sgmnt No.	Dist. Below Gages ft	max. Force kips	min. Force kips	max. Comp. Stress ksi	max. Tens. Stress ksi	max. Trnsfd. Energy kip-ft	max. Veloc. ft/s	max. Displ. in
1	3.3	849.2	0.0	36.7	0.00	76.90	17.9	1.436
2	6.6	858.9	0.0	37.1	0.00	75.99	17.7	1.400
4	13.1	812.6	0.0	35.1	0.00	69.73	17.3	1.328
6	19.7	792.5	0.0	34.3	0.00	65.58	17.1	1.253
8	26.3	801.5	0.0	34.7	0.00	62.96	16.7	1.177
10	32.8	808.2	0.0	34.9	0.00	59.68	16.2	1.100
12	39.4	791.5	0.0	34.2	0.00	54.87	15.5	1.023
14	45.9	759.3	0.0	32.8	0.00	49.26	14.8	0.944
16	52.5	725.7	0.0	31.4	0.00	43.63	14.1	0.862
18	59.1	709.8	0.0	30.7	0.00	38.41	13.4	0.781
20	65.6	705.6	0.0	30.5	0.00	33.58	12.5	0.700
22	72.2	707.3	0.0	30.6	0.00	28.64	11.6	0.621
24	78.8	693.8	0.0	30.0	0.00	23.67	10.6	0.544
26	85.3	669.9	0.0	29.0	0.00	19.10	9.7	0.470
28	91.9	643.6	0.0	27.8	0.00	15.15	8.8	0.399
30	98.5	613.8	0.0	26.5	0.00	11.72	7.8	0.330
32	105.0	586.3	0.0	25.4	0.00	8.72	6.7	0.264
34	111.6	545.7	0.0	23.6	0.00	6.08	5.5	0.202
36	118.2	485.1	0.0	21.0	0.00	3.82	4.4	0.145
37	121.4	485.7	0.0	21.0	0.00	3.29	3.8	0.118
38	124.7	406.2	0.0	17.6	0.00	2.05	3.3	0.095
39	128.0	408.1	0.0	17.6	0.00	1.05	2.6	0.072
Absolute	9.8			37.4			(T =	22.1 ms)
	3.3				0.00		(T =	0.0 ms)

CASE METHOD

J =	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
RP	1147.3	1103.0	1058.6	1014.2	969.9	925.5	881.1	836.8	792.4	748.1
RX	1147.3	1103.0	1058.7	1014.9	971.1	927.3	883.5	839.7	795.9	752.1
RU	1200.6	1161.6	1122.6	1083.5	1044.5	1005.5	966.4	927.4	888.4	849.3

RAU = 159.7 (kips); RA2 = 844.9 (kips)

Current CAPWAP Ru = 880.2 (kips); Corresponding J(RP)= 0.60; J(RX) = 0.61

VMX	TVP	VT1*Z	FT1	FMX	DMX	DFN	SET	EMX	QUS
ft/s	ms	kips	kips	kips	in	in	in	kip-ft	kips
17.91	21.48	739.0	851.9	851.9	1.423	0.067	0.077	77.1	1233.8

PILE PROFILE AND PILE MODEL

Depth	Area	E-Modulus	Spec. Weight	Perim.
ft	in^2	ksi	lb/ft^3	ft
0.00	23.12	29992.2	492.000	5.236
128.00	23.12	29992.2	492.000	5.236

Toe Area 2.182 ft^2

Top Segment Length 3.28 ft, Top Impedance 41.27 kips/ft/s

Pile Damping 1.0 %, Time Incr 0.195 ms, Wave Speed 16807.9 ft/s, 2L/c 15.2 ms

APPENDIX F

GLOBAL STABILITY ANALYSIS RESULTS

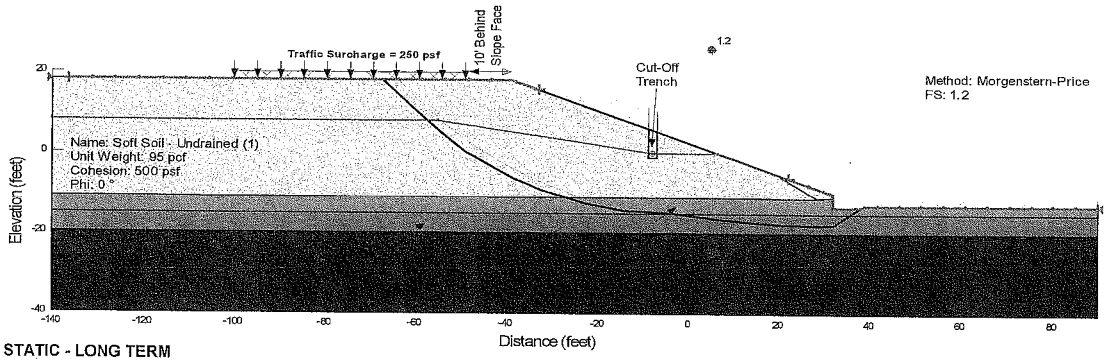
APPENDIX F

GLOBAL STABILITY ANALYSIS RESULTS

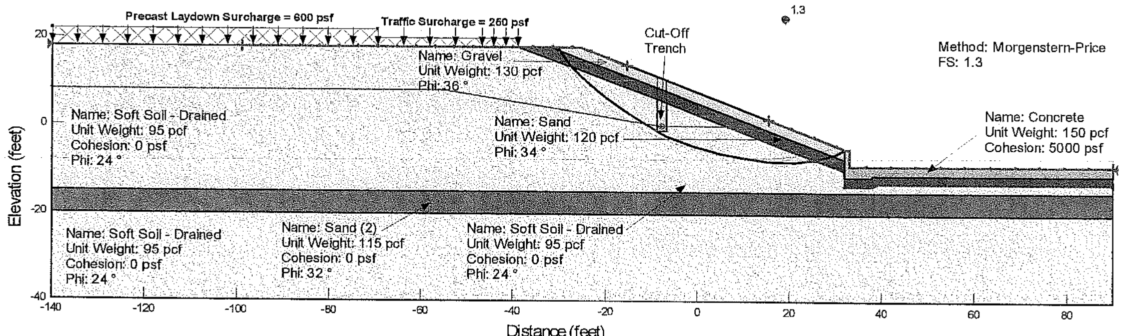
TABLE OF CONTENTS

FIGURES

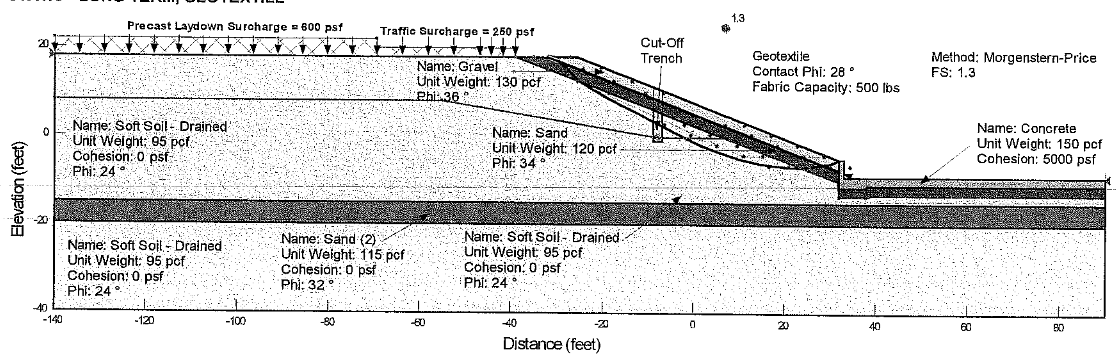
STATIC - CONSTRUCTION



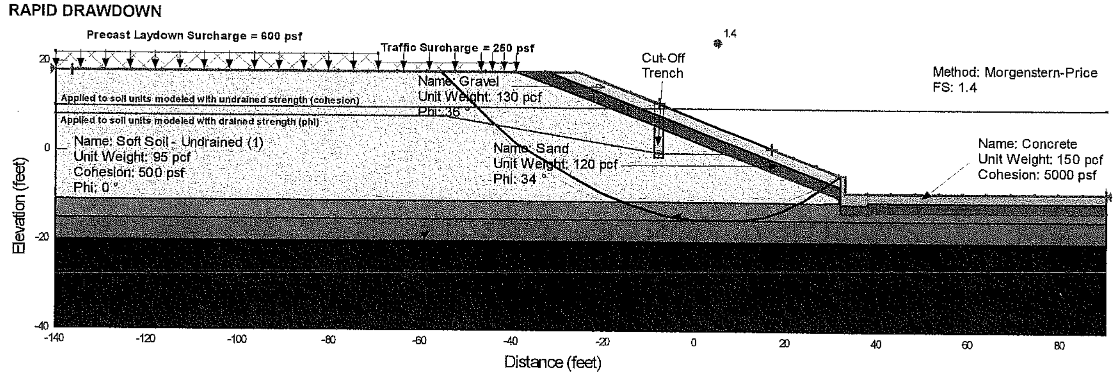
STATIC - LONG TERM



STATIC - LONG TERM, GEOTEXTILE



RAPID DRAWDOWN



NOTES:

1. Thick failure surface line corresponds to the critical optimized failure surface. Thin failure surface line corresponds to the critical circular failure surface.
2. These analyses show a simplification of the generalized slope and do not show crane trestle beam which would intersection the native ground surface at the top of the slope. See Table 2 for a schematic drawing of the generalized top of the basin slope.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

GLOBAL STABILITY ANALYSES RESULTS
BASIN SLOPE NORTH 2.5H:1V
HEIGHT = 27 FEET

December 2010 | 21-1-21190-015

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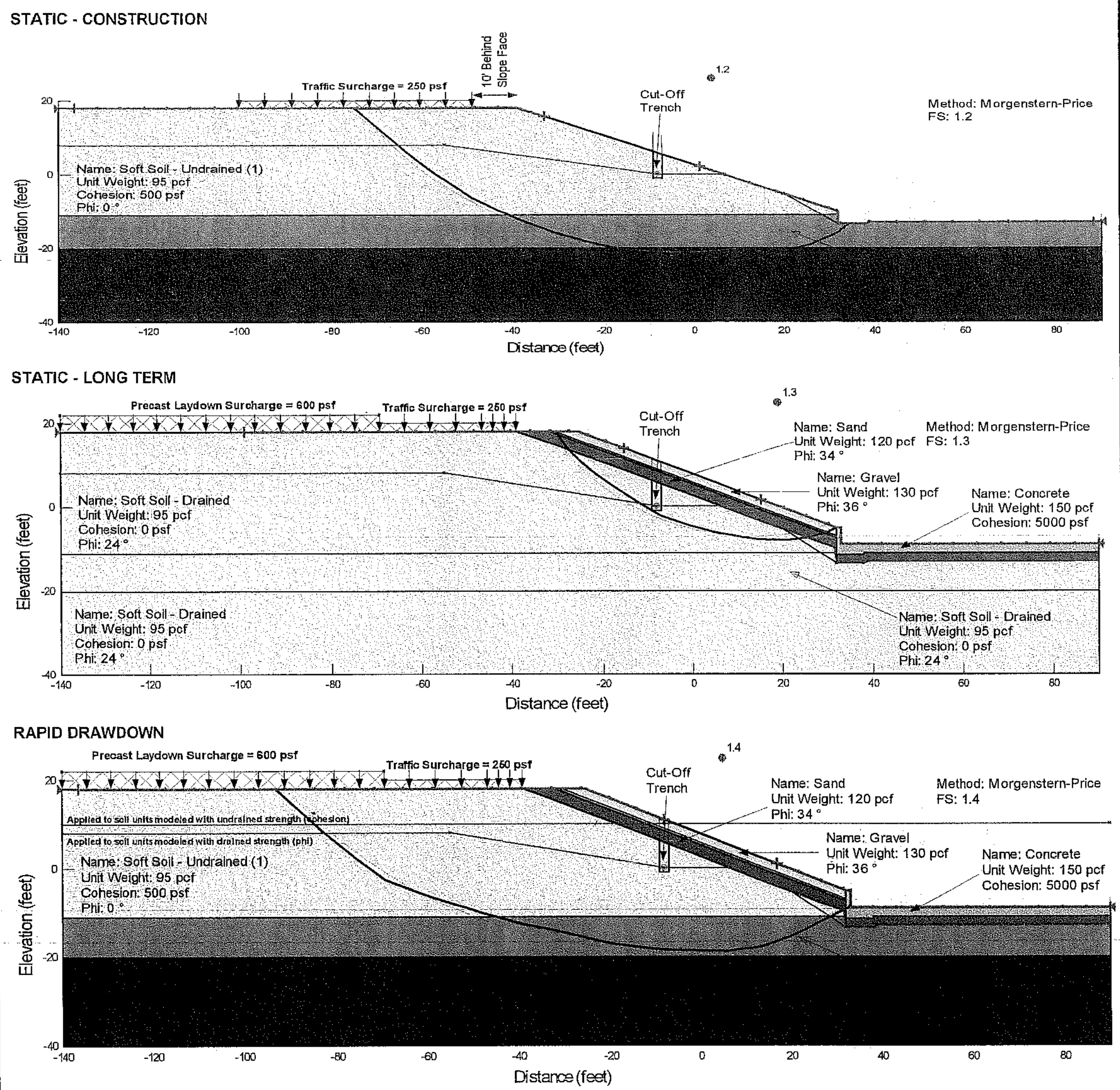
Geotechnical and Environmental Consultants

FIG. F-1

FIG. F-1



NOTES:

1. Thick failure surface line corresponds to the critical optimized failure surface. Thin failure surface line corresponds to the critical circular failure surface.
2. These analyses show a simplification of the generalized slope and do not show crane trestle beam which would intersection the native ground surface at the top of the slope. See Table 2 for a schematic drawing of the generalized top of the basin slope.

FIG. F-2

SR 520 Pontoon Casting Facility
Aberdeen, Washington

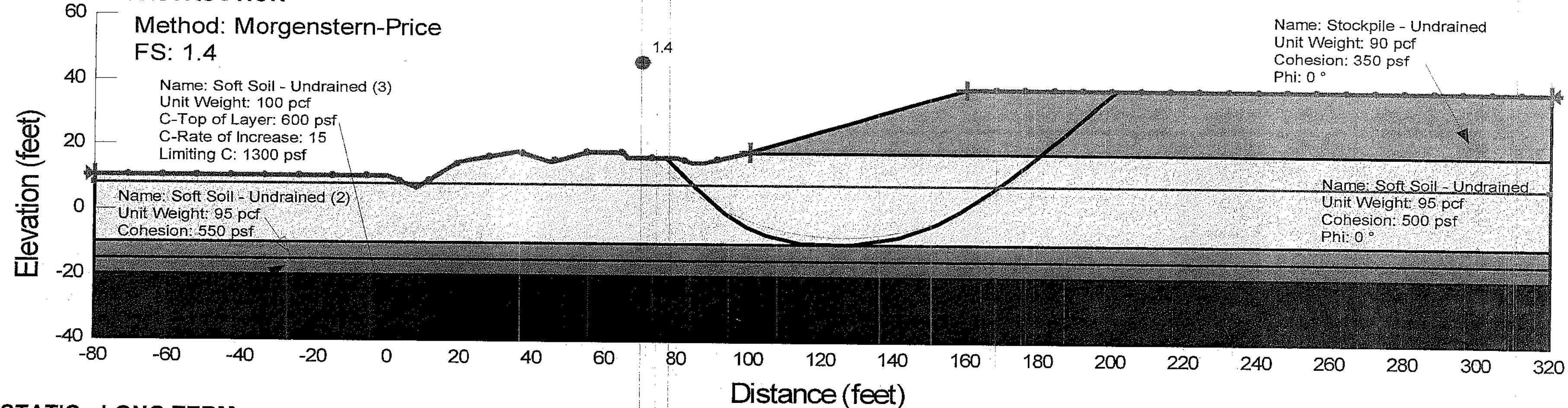
GLOBAL STABILITY ANALYSES RESULTS
BASIN SLOPE SOUTH 2.5H:1V
HEIGHT = 27 FEET

December 2010 | 21-1-21190-015

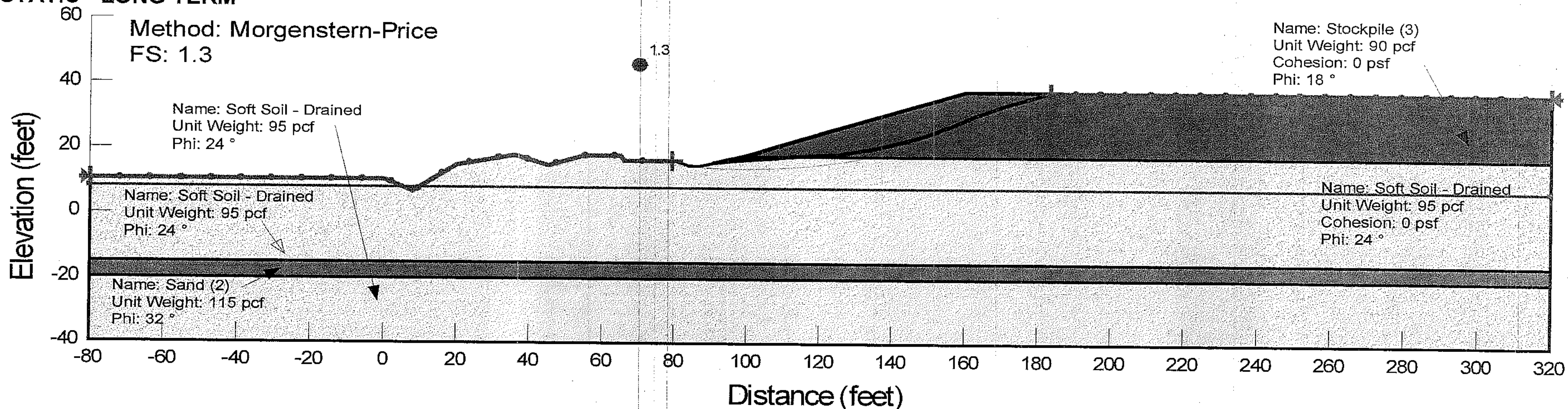
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. F-2

STATIC - CONSTRUCTION



STATIC - LONG TERM



Notes

1. Thick failure surface line corresponds to the critical optimized failure surface. Thin failure surface line corresponds to the critical circular failure surface.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

GLOBAL STABILITY ANALYSES RESULTS
STOCKPILE SLOPE 3H:1V
HEIGHT = 20 FEET

December 2010 21-1-21190-015

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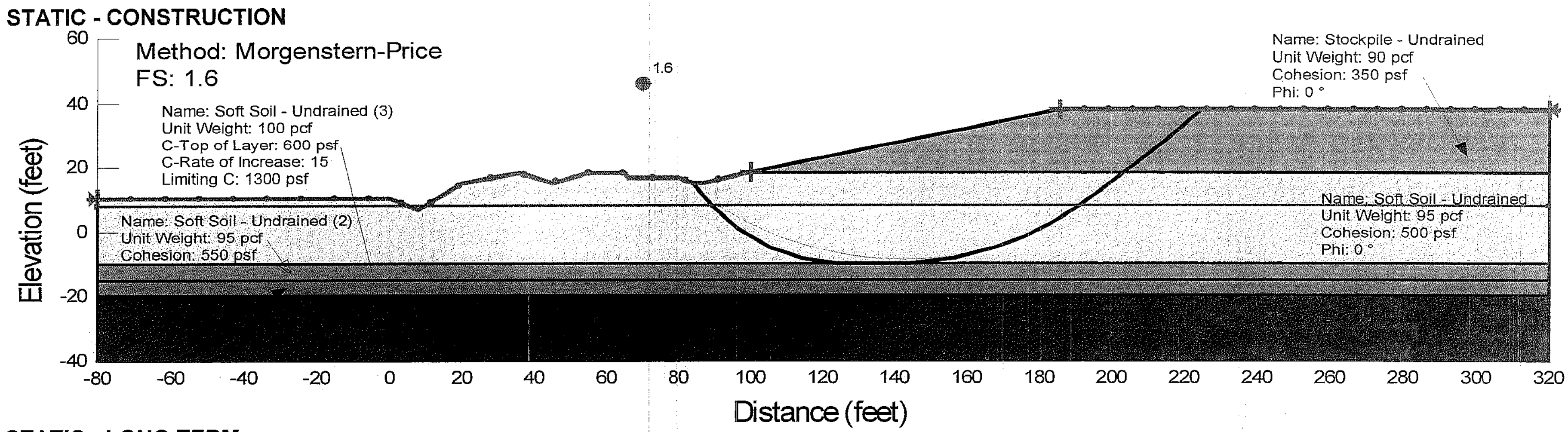
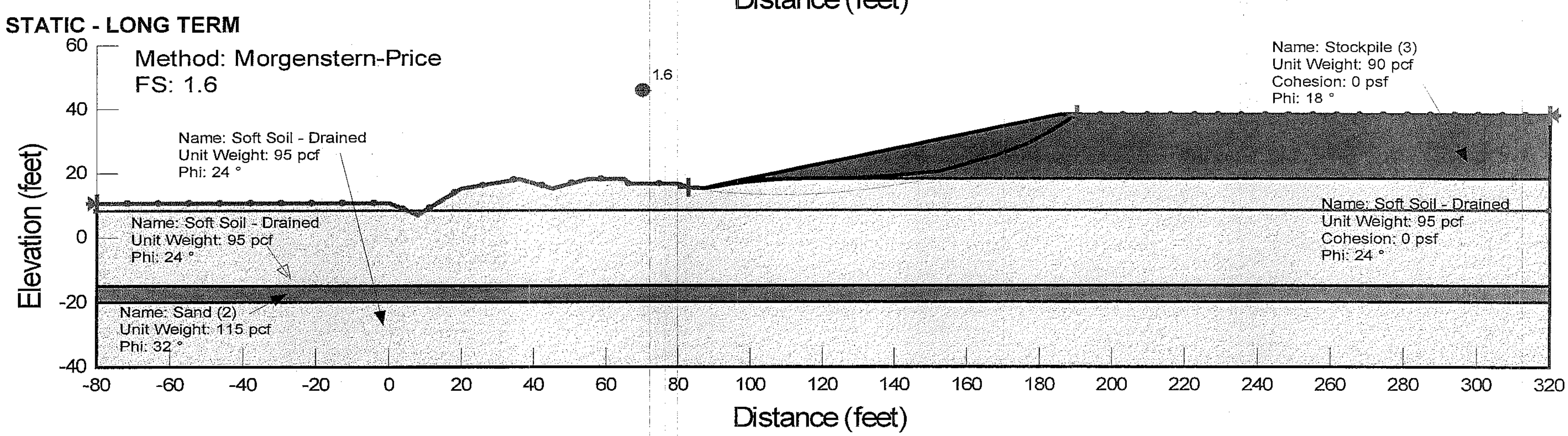
FIG. F-3

FIG. F-3





Notes

1. Thick failure surface line corresponds to the critical optimized failure surface. Thin failure surface line corresponds to the critical circular failure surface.

FIG. F-4

SR 520 Pontoon Casting Facility
Aberdeen, Washington

GLOBAL STABILITY ANALYSES RESULTS
STOCKPILE SLOPE 4H:1V
HEIGHT = 20 FEET

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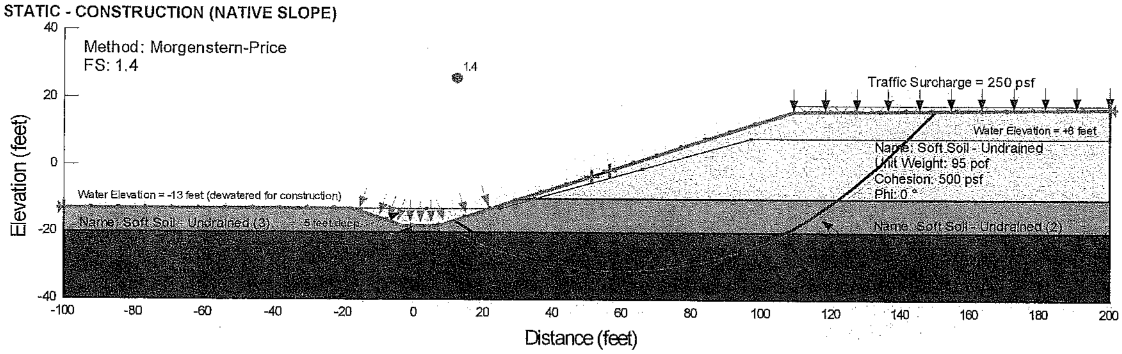
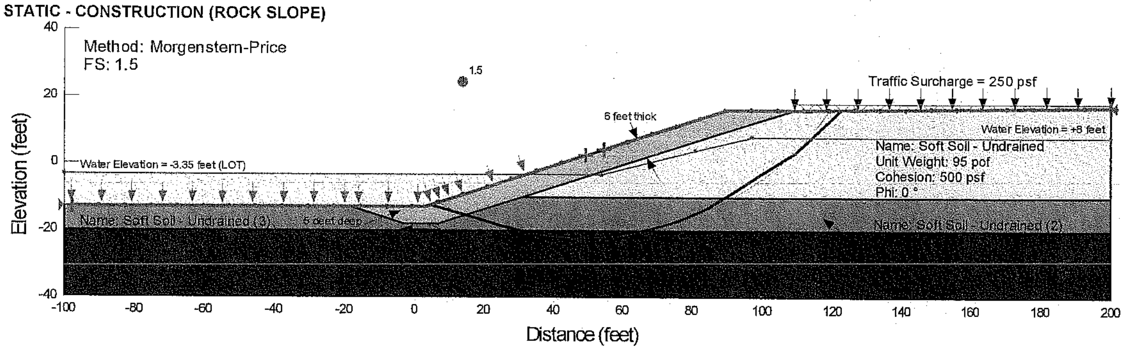
FIG. F-4

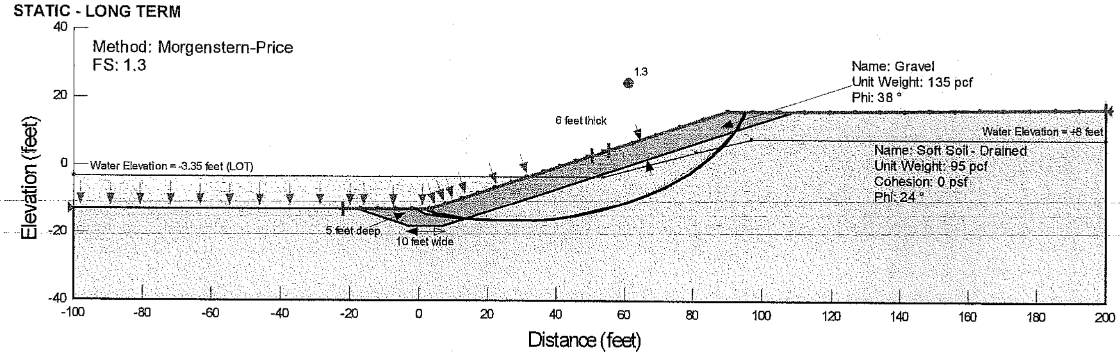
STATIC - CONSTRUCTION (NATIVE SLOPE)



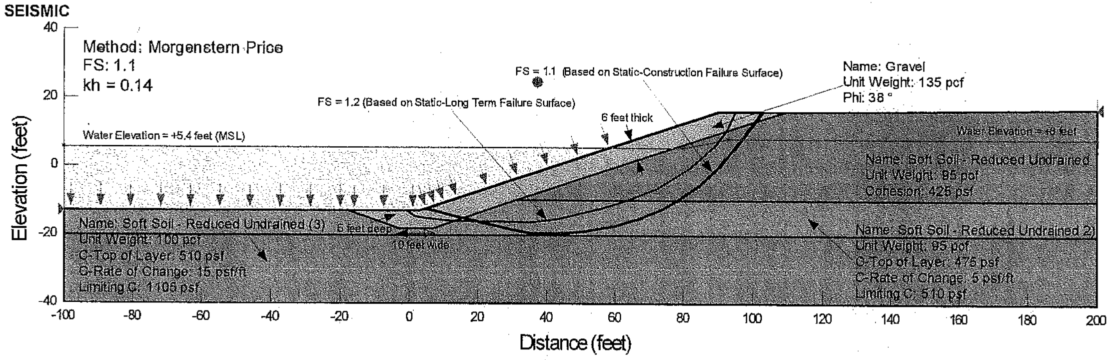
STATIC - CONSTRUCTION (ROCK SLOPE)



STATIC - LONG TERM



SEISMIC



NOTES:

1. Static Cases:
   Thick failure surface line corresponds to the critical optimized failure surface.
   Thin failure surface line corresponds to the critical circular failure surface.

2. Seismic Case:
   The seismic factor of safety is estmiated by applying the horizontal coefficient, kh, to the pre-determined Static - Long Term and Static - Construction failure surfaces.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

GLOBAL STABILITY ANALYSES RESULTS
LAUNCH CHANNEL SLOPE 3H:1V
HEIGHT = 29 FEET

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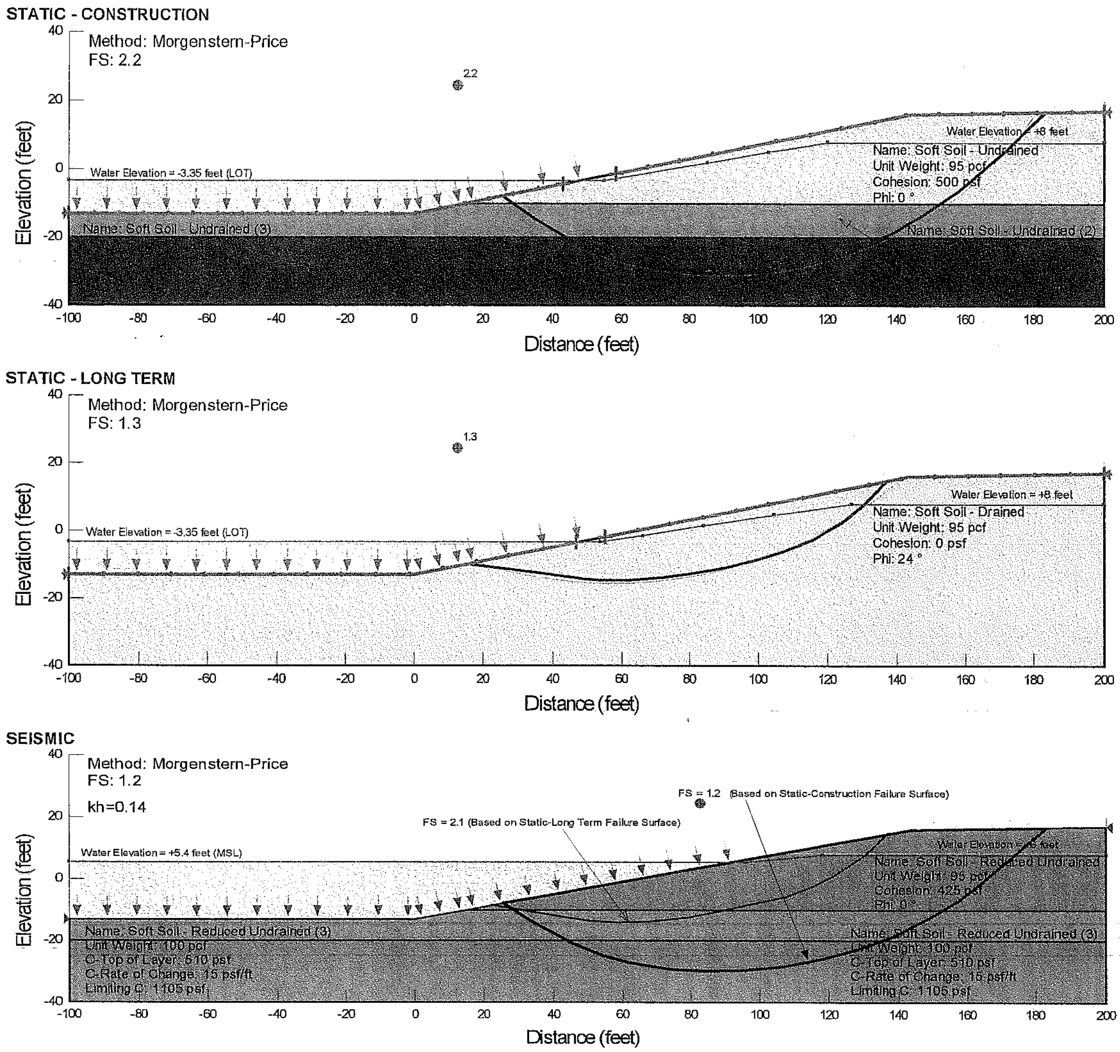
FIG. F-5



NOTES:

1. Static Cases:
   Thick failure surface line corresponds to the critical optimized failure surface.
   Thin failure surface line corresponds to the critical circular failure surface.

2. Seismic Case:
   The seismic factor of safety is estmiated by applying the horizontal coefficient, kh, to the pre-determined Static - Long Term and Static - Construction failure surfaces.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

GLOBAL STABILITY ANALYSES RESULTS
LAUNCH CHANNEL SLOPE 5H:1V
HEIGHT = 29 FEET

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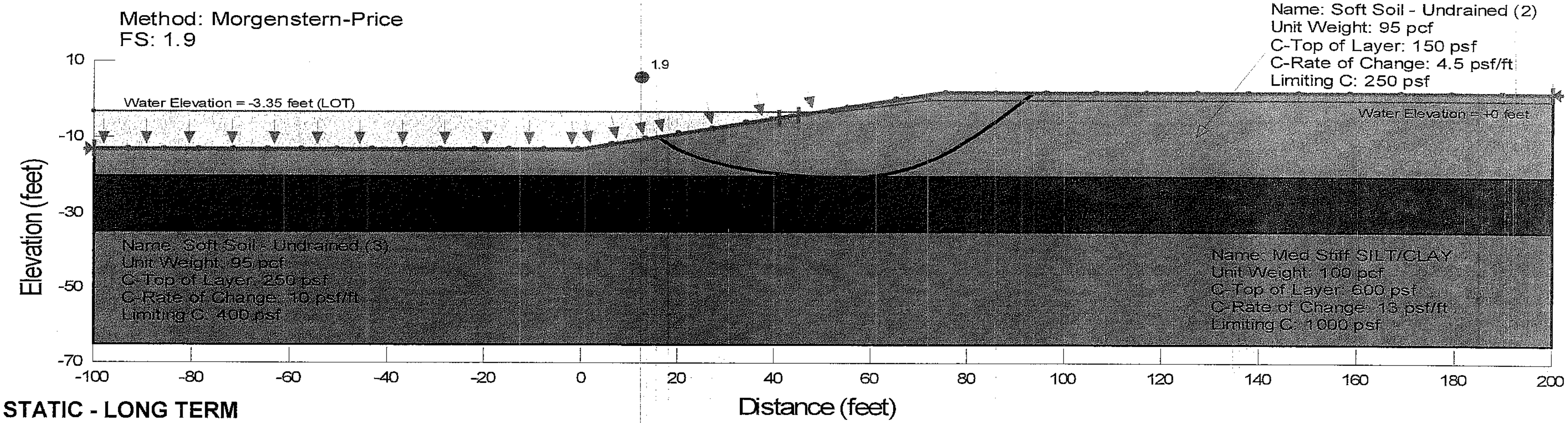
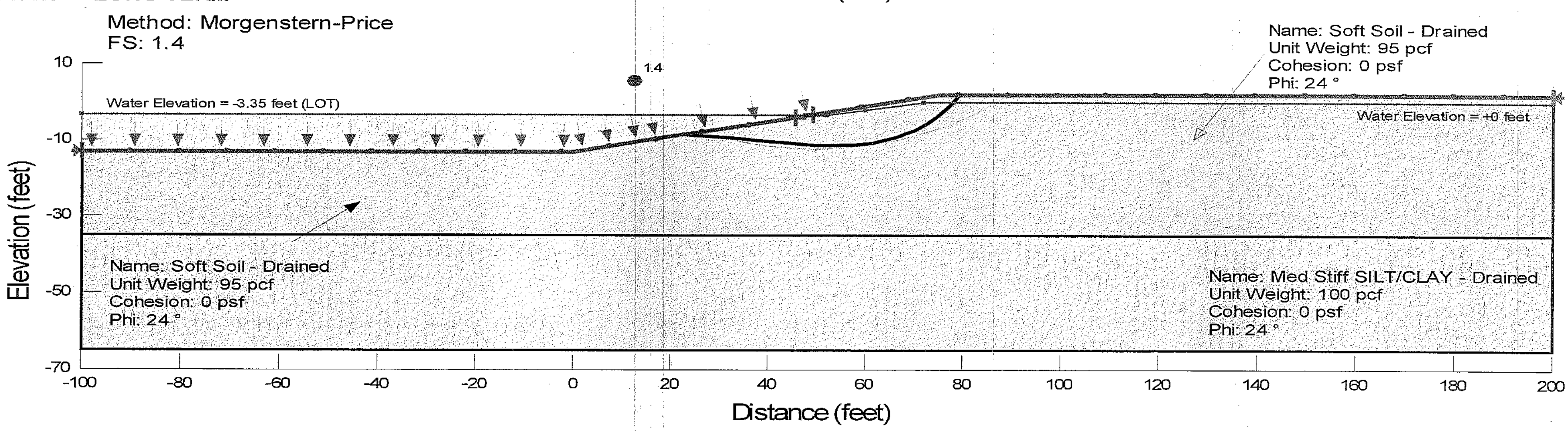
FIG. F-6

FIG. F-6

STATIC - CONSTRUCTION



STATIC - LONG TERM



Notes

1. Thick failure surface line corresponds to the critical optimized failure surface. Thin failure surface line corresponds to the critical circular failure surface.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

**GLOBAL STABILITY ANALYSES RESULTS
LAUNCH CHANNEL SLOPE 5H:1V
HEIGHT = 15 FT, OFFSHORE PROFILE**

December 2010 21-1-21190-015

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FIG. F-7

FIG. F-7

APPENDIX G

ONE- AND TWO-DIMENSIONAL GROUND RESPONSE

APPENDIX G

ONE- AND TWO-DIMENSIONAL GROUND RESPONSE

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TABLES (cont.)

FIGURES

FIGURES (cont.)

APPENDIX G

ONE- AND TWO-DIMENSIONAL GROUND RESPONSE

G.1 GENERAL

We performed two types of site response analyses to estimate the soil response during the design ground motion: one-dimensional (1D) equivalent linear total stress analysis and two-dimensional (2D) non-linear effective stress analysis. The 1D equivalent linear total stress analysis is a method to estimate site response for soil profiles where pore pressure generation is not considered. Although site soil profiles would generate excess pore pressure during strong ground shaking, the 1D equivalent linear total stress analysis would provide relatively higher ground motions as compared to a 1D or 2D non-linear effective stress analysis.

Evaluations of site-specific non-linear 2D soil response including the effects of dynamic pore pressure generation were performed to evaluate the generation of excess pore pressure, soil softening, and lateral ground displacement effects on the Pontoon Casting Facility (PCF). 2D models were selected to evaluate the ground response in both the transverse (east-west) and longitudinal (north-south) axes of the PCF. A description of the methods, inputs, and results is presented below.

G.2 METHODS

The following steps were performed to obtain 1D and 2D site response of the subsurface soils during ground shaking:

(1) Spectrally match seven acceleration time histories so that their corresponding spectra accelerations match the 975-year uniform hazard spectrum.

(2) Develop 1D and 2D stratigraphy, strength, and shear wave velocity profiles. The 1D site response analyses were performed for the borings that had measured shear wave velocity. The 2D site response analyses were performed using representative profiles presented in Appendix D.

(3) Perform site response. Results of analysis are given with respect to depth and time for parameters such as: acceleration (1D, 2D), soil displacement (2D), pile displacement, and moment (2D transverse).

These steps are discussed in greater detail below.

G.3 UNIFORM HAZARD GROUND MOTION

A soft rock uniform hazard spectrum (soft rock target) is required for the development of the design ground motion time histories. The soft rock level motion uniform hazard spectrum (UHS) was obtained from the 2002 U.S. Geological Survey (USGS) National Seismic Hazard Mapping Program probabilistic seismic hazard analyses (PSHA) by Frankel and others (2002), and is based on ground motions consistent with those used in the development of the American Association of State Highway and Transportation Officials (AASHTO, 2008) ground shaking hazard maps and design tool (i.e., 975-year return period). The 975-year UHS was obtained from the USGS web site using the latitude and longitude of the site. Figure G-22 presents the soft rock UHS used for the project.

G.4 DEVELOPMENT OF ROCK INPUT MOTIONS

We used deaggregation results from the USGS PSHA performed for this site to guide the selection of input time histories. The deaggregation results provide earthquake magnitude and distances that are the most significant contributors to ground motion hazard for a particular return period and spectral acceleration period.

For ground motions with a 975-year return period, the main contributors of seismic hazard include mega-thrust interface earthquakes on the Cascadia Subduction Zone. The characteristic magnitude and distance for the main seismogenic source contributors in the USGS PSHA are magnitude 8.3 and a source distance of approximately 20 kilometers. We searched publicly available ground motion databases for previously recorded earthquake motions with characteristics similar (i.e., tectonic source, magnitude, distance, etc.) to those identified in the seismic hazard deaggregation. Table G-1 lists the candidate reference recorded earthquake motions that were considered.

A candidate reference time history is a recorded earthquake time history that is to be considered for spectral matching to produce a spectrum-compatible time history. Ideally, a natural time history, or reference motion, would be from a recorded earthquake of similar magnitude, fault type, and tectonic regime, and be the same distance from and have the same site rock conditions as the soft rock UHS (i.e., soft rock conditions from the ground motion attenuation equations that were used in the PSHA). Although these characteristics cannot be exactly matched, candidate reference ground motions were obtained from previously recorded subduction zone earthquakes at locations worldwide that, to the extent possible, had similar source characteristics.

To develop the preferred list of reference ground motions, when available we preferred subduction ground motions with scaled response spectra near the target soft rock UHS. Ideally, during the spectral matching process, we prefer to remove energy from the time history (i.e., scale down) ground motions than add energy (i.e., scale up). This approach was not possible for this project, given that suitable recorded subduction zone earthquake time histories with short source-to-site distances and soft rock site conditions were not available.

We also reviewed magnitude, distance, Arias Intensity, duration, and other parameters in our time history selection evaluation. Given the limited database of subduction zone earthquake time histories that yield reasonable time histories, the seven preferred reference recorded earthquake motions presented in Table G-2 were selected. Figures G-1 through G-7 presents the unscaled time histories and response spectra for the selected reference time histories. These unscaled soft rock response ground motions correspond to the time histories as they were originally recorded. A total of three earthquake events are represented, while four of the time histories are from the same earthquake event. These time histories provide the preferred matching characteristics to the target soft rock UHS.

Spectrum-compatible rock time histories were developed using the program RSPMATCH (Abrahamson, 1994) and BLINE (Abrahamson personal communication) to spectrally match and baseline correct selected recorded (reference) earthquake motions to the UHS representative of design ground motion. RSPMATCH performs spectral matching in the time domain by adding wavelets to the initial time history. Figures G-8 through G-14 present the matched soft rock time histories and response spectra that correspond to the reference time histories and the site-specific UHS. The significant durations of the matched seven time histories vary between 20 and 55 seconds with an average of about 35 seconds. By matching to the soft rock uniform hazard target spectrum, we are matching to a spectrum that includes the significant earthquake magnitudes and source-to-site distances in the PSHA. Therefore, by matching to the project soft rock uniform hazard target spectrum and utilizing seven representative recorded subduction zone ground motions, the significant durations of the preferred seven time histories correspond to significant earthquake magnitudes and source-to-site distances in the PSHA.

G.5 ONE-DIMENSIONAL (1D) EQUIVALENT LINEAR SITE RESPONSE ANALYSIS

G.5.1 Shear Wave Velocity Profiles

We used measured shear wave velocity profiles from borings BH-1-10, BH-2-10, H-07-09, H-08-09, H-16-09, and H-18P-09 to generate best-estimate shear wave velocity profiles for our site response model. We also used the soil profiles from the boring logs to estimate the soil properties for each soil layer. Soil properties for each layer included soil unit weight, shear modulus versus strain curves, and damping versus strain curves. The modulus degradation and damping curves we used include EPRI Sand (EPRI, 1993), EPRI Rock (EPRI, 1993), Gravel (Rollins and others, 1998), and Clay Plasticity Index (PI) = 15 (Vucetic and Dobry, 1991) (Figure G-15). Figures G-16 through G-21 present the shear wave velocity and soil property profiles for each soil boring.

G.5.2 Methods

We used the computer program FLAC 6.0 (Itasca, 2008) to perform one-dimensional total stress site response analyses. FLAC is a finite difference program that simulates continuous materials such as soil. The finite difference formulation allows FLAC to explicitly model site response in the time domain.

The 1D soil models comprised a column of 1-foot-square soil zones. The vertical and horizontal stresses in the soil column were based on the soil unit weights and an assumed horizontal stress coefficient of 0.5 for normally consolidated soils (sand and silts) and 1.0 for overconsolidated or dense soils (dense gravels and siltstone).

Because the input motions are assumed to originate from upward propagating waves (i.e., outcrop motions), we applied FLAC's quiet (compliant) base formulation to the bottom of the model so that downward-propagating waves would not reflect off the model boundaries.

To model the soil stress-strain behavior under dynamic loading, we used FLAC's elastic constitutive model coupled with the hysteretic model. Elastic model input includes soil density (unit weight), shear modulus, and Poisson's ratio. We calculated the shear modulus based on the soil density and shear wave velocity, and Poisson's ratio.

We used the hysteretic model to represent the shear modulus degradation and damping observed during cyclic shearing of soil. The hysteretic model requires the shear modulus versus shear strain curve of the soil to be fit to closed-form equations available in FLAC. We matched the FLAC equations to the curves in Figure G-15. Results of the matching process are shown in Figures G-32 through G-36.

We performed the total stress analysis for each of the seven time histories and each of the six borings. After each analysis, we extracted the ground surface velocity time history, differentiated the time history, and calculated an acceleration response spectrum.

G.5.3 Results

Figures G-22 through G-27 show the individual ground surface acceleration response spectra for borings BH-1-10, BH-2-10, H-07-09, H-08-09, H-16-09, and H-18P-09 considering all seven time histories. Also plotted in each figure are the USGS UHS and the AASHTO Site Class E spectrum for the project site. Figure 15 in the main text presents the geometric mean of the response spectra from each boring and the recommended design response spectrum. The recommended design response spectra can be used for dynamic inertial analyses.

G.6 TWO-DIMENSIONAL EFFECTIVE STRESS SITE RESPONSE

G.6.1 Model Setup

G.6.1.1 Longitudinal

The longitudinal models represent the idealized design 2D cross sections in the north-south direction. Five longitudinal cross sections were evaluated along the length of the basin based on the four cross sections in Figure D-1 in Appendix D. Model cross sections along the centerline of the basin and outside the basin (at the top of the slope to the west) are shown in Figures G-28 and G-29, respectively. These model cross sections are generalized from Sections A-A' and B-B' as shown in Figure D-1. These models do not include soil-structure interaction effects and represent free-field response.

G.6.1.2 Transverse

The transverse models represent idealized design 2D cross sections in the east-west direction. Two transverse cross sections were evaluated across the basin. The transverse models also include the soil structure interaction effects of the crane trestle and basin slab/foundation structures. The northern model cross section, shown in Figure G-30, considers the level ground conditions outside the basin to the east and west along with a shallow sand layer between elevations -20 to -30 feet. This model is generalized from F-F' as shown in Figure D-2 and borings to the east and west and is intended to represent the basin conditions from a east-west line 300 feet north of the gate and points further north.

The southern model cross section, shown in Figure G-31, considers level ground conditions outside the basin to the east and a 20-foot-high stockpile to the west and is intended to represent the basin conditions between the gate and an east-west line 300 feet north of the gate. This model is generalized from sections G-G' and H-H' as shown in Figure D-2 and borings to the east and west. Based on these sections and borings, the shallow sand layer was only considered to be present beneath the stockpile and was not extended to the basin.

G.6.2 Constitutive Models

Constitutive models use differential equations to describe stress-strain relationships of a material such as soil. FLAC provides several internal constitutive models including the elastic, Mohr-Coulomb, and hysteretic model. The elastic and Mohr-Coulomb models combined with the hysteretic model were used to model soil at the site that was assumed to not exhibit liquefied soil behavior. A user-defined constitutive model developed by the University of British Columbia, UBCSAND, Byrne and others (2004) was used on soil where significant pore pressure changes are anticipated during dynamic loading of design ground motions. A summary of the constitutive models used is presented in Table G-3. A brief discussion of these models is provided below.

G.6.2.1 Elastic and Mohr-Coulomb

The Mohr-Coulomb model treats a material as linear-elastic-purely-plastic. That is, the model behaves as a linearly elastic material at shear stresses less than the prescribed yield shear strength. When shear stress demands reach and remain at the yield strength, permanent shear strains will develop.

The properties required for the Mohr-Coulomb model, as implemented in FLAC, include mass density, cohesion, angle of internal friction, tension limit, dilation angle, bulk modulus, and shear modulus. Mass density represents the mass of the soil; cohesion, angle of internal friction, tension limit, and dilation angle describe the shear strength limit of the soil; and bulk and shear modulus describe the elastic behavior of the soil. It was anticipated that the dense gravels and siltstone would not reach their strength limits during dynamic loading; therefore, to reduce computational runtimes, FLAC's elastic model was used for these soil units. FLAC's elastic model behaves in a manner similar to that of the Mohr-Coulomb model without a strength limit or ability to accumulate plastic deformations; thus, no strength parameters are needed. The soil properties used in the analyses are described in the next section.

G.6.2.2 Hysteretic Model

Soil experiencing large load ranges and reversals exhibits hysteretic behavior; that is, the shear modulus decreases and damping increases with increasing shear strain. Upon a strain reversal, the modulus and damping return to their low strain values and the modulus reduction and damping increase start over. Since the Mohr-Coulomb and elastic soil models do not alone model this behavior, FLAC's hysteretic model was also used. The hysteretic model requires the shear modulus versus shear strain curve of the soil to be fit to one of four closed-form equations available in FLAC. To match the damping versus strain behavior, the area under the fit stress-strain curve, which describes the amount of damping the model will exhibit under a given strain loading, was monitored. The fitting parameters were iterated during the hysteretic curve fitting process so that a good match of modulus reduction and damping at the anticipated strain levels could be made. The results of the modulus reduction and damping curve fitting process are shown in Figures G-32 through G-36. As can be seen in these figures, the damping at very low strains is underestimated; therefore, 0.2 percent Rayleigh damping was added to the model to sufficiently dissipate energy at small strains.

G.6.2.3 Calibration for Plasticity Index (PI) <17 Silts

The FLAC hysteretic damping and modulus reduction model was used to approximate the dynamic behavior of Silts with a PI less than17. The anticipated dynamic behavior was evaluated based on the 2D site response results, cyclic direct simple shear (CDSS) test results and our experience. The maximum change in the horizontal shear stress during dynamic loading, or maximum cyclic shear stress, was recorded during the 2D model runs. These values were multiplied by 0.65 to approximate a uniform cyclic shear stress for the input ground motions that represent Mw = 8.3 (23 cycles), which could be directly compared to the CDSS tests. Multiplying 0.65 by the average of the maximum shear stress for all motions was approximately equal to 0.25. Results of the CDSS tests with PIs less than 17 and a cyclic stress ratio (CSR) = 0.25 cycled for 23 cycles indicates maximum shear strains that range from 0.5 to 2.5 percent and excess pore pressure ratios between 0.16 and 0.68. These results are reprinted from the reference documents in Figure G-44. Based on these results, the calibration of the numerical model was focused on achieving an approximate shear strain magnitude of 2 percent under a CSR = 0.25 which represents the upper bound of expected shear strain under a loading of

CSR = 0.25 at 23 cycles. The Vucetic and Dobry modulus reduction curves closely match this criterion. The results of the calibration compared to the CDSS tests are shown in Figure G-44.

G.6.2.4 UBCSAND Calibration for Loose to Medium Dense Sands

The UBCSAND constitutive model was developed by Professor Peter M. Byrne and his colleagues at the University of British Columbia, Vancouver, Canada. UBCSAND modifies the internal Mohr-Coulomb model in FLAC to better capture the plastic strain response of the soil at most stages of loading and unloading. In addition, the UBCSAND model uses a hyperbolic formulation to describe the shear and bulk modulus of the material as a function of the current effective stresses and any changes during loading. These additional features allow the model to approximate the non-linear hysteretic behavior which is observed in loose granular soil that develops significant pore pressures.

The UBCSAND model as implemented in FLAC requires a total of twelve input and four calibration parameters. Most of the input parameters can be empirically related to other parameters, therefore, only a few parameters are required. The input parameters and their relation to each other are described briefly below:

- $(N_1)_{60}$: Standard Penetration Test blow count corrected to 60 percent hammer efficiency and 1 ton per square foot of overburden pressure.
- ϕ_{CV}: Constant volume friction angle is used to describe the transition from dilative to contractive soil behavior. This parameter was set to a value of 33 degrees, which is typical for Fraser River sands.
- ϕ_F: Failure friction angle was set based on the following empirical relationship typically used with UBCSAND to model Fraser River sands:

$$\emptyset_f = \emptyset_{CV} + \frac{(N_1)_{60}}{10}$$

- *kGe, ne*: Shear modulus number and exponent used to describe shear modulus at different effective confining stresses.

$$Gmax = \rho \times Vs^2;\ Gmax = kGe \times Pa \times \left(\frac{\sigma_m{}'}{Pa}\right)^{ne};\ kGe = 21.7 \times Y \times ((N_1)_{60})^{0.33}$$

Based on our experience, "Y" and "ne" values of 12 and 0.52, respectively, were chosen to best represent the site soil.

- *k*B, me: Bulk modulus number and exponent used to describe bulk modulus at different effective confining stresses. The bulk modulus was calculated using elastic equations and assuming a Poisson's ratio of 0.1. The bulk modulus exponent was set equal to the shear modulus exponent.
- *k*Gp, np: Plastic shear modulus number and exponent used to describe plastic shear modulus at different effective confining stresses. These values were calculated based on the following empirical equation typically used with UBCSAND loose sands:

$$kGp = 0.003 \times kGe \times ((N_1)_{60})^2 + 75; \; np = 0.4$$

- R_f: Hyperbolic failure ratio that was set based on the following empirical relationship typically used with UBCSAND to model loose sand:

$$R_F = 1 - \frac{((N_1)_{60})}{100}$$

The four calibration parameters of the UBCSAND model used to calibrate the pore pressure generation and post-liquefied soil behavior are described below:

- "hfac1" and "hfac2": These parameters control the pore pressure generation versus number of constant strain cycles (Ncycles) relationship. In our experience, "hfac1" is the primary parameter that controls this relationship while "hfac2" modifies the shape of the curve; therefore, "hfac2" was set to a value of "1" and not varied in the calibration.
- "*hfac3*": This parameter controls a portion of the shear stress versus shear strain curve immediately after the shear stress reverses sign. "*hfac3*" was set to a value of "1" and not varied in the calibration.
- "*hfac4*": This parameter controls the dilative behavior as a function of the Ncycles experienced. "*hfac4*" was set such that the maximum shear strain in a cyclic direct simple shear simulation would approximate the findings of Seed and others (1985).

We calibrated UBCSAND to closely replicate the empirical liquefaction triggering behavior described in Youd and others (2001). The number of cycles (Ncycles) to reach liquefaction was assumed to occur at an excess pore pressure ratio (Ru) equal to 0.9. The specific Youd and others (2001) empirical relationships targeted in our calibration include the cyclic resistance ratio (CRR) versus $(N_1)_{60cs}$ at 15 cycles, CRR versus effective confining stress (Kσ effects), and CRR versus Ncycles based on site-specific CDSS testing (Magnitude effects). The calibration process is an iterative process where the "hfac1" parameter value that simulates the desired behavior at various $(N_1)_{60}$ and confining stresses is determined. To choose a value for "*hfac1*," a CDSS test is modeled in FLAC. "*hfac1*" is adjusted while holding $(N_1)_{60}$ and confining stress constant until the Ru in FLAC reaches a value of 0.9 after 10 constant strain cycles. A comparison of the calibrated UBCSAND output to the target Youd and others (2001) empirical relationships is shown in Figures G-37 and G-38.

A series of single-element simulations were performed to evaluate these observations. Shear stress time series were input into single-element simulations to evaluate UBCSAND's predictions under various loading scenarios. Results of one of these simulations compared to published laboratory test data for loose sands subject to similar shear stress loadings are shown in Figure G-39. It can be seen that UBCSAND does not consistently track non-uniform stress reversals. Based on our review and testing of the UBCSAND source code, it is our opinion that this inconsistency is a result of the strain reversal logic and the association of strain predictions with accumulated shear strain. The logic essentially treats all cycles, even small shear stress cycles, as if a large shear stress cycle has been completed, which potentially results in an over-prediction of the shear strain in the next loop. This behavior accumulates and results in predicted shear strains that are potentially orders of magnitude too large.

In consideration of the above, the Mohr-Coulomb constitutive model was used to approximate the expected post-liquefied behavior. Each zone in the model was monitored throughout the dynamic simulation for shear strains that exceeded an absolute value of 3.75 percent. A threshold shear strain of 3.75 percent is commonly used as the definition of when "liquefaction" occurs and is consistent with the development of empirical liquefaction correlations. When the shear strain in a zone exceeded this threshold, the constitutive model was changed to the Mohr-Coulomb constitutive model with residual sand parameters. The residual strength was linearly interpolated between the static strength at a $(N_1)_{60cs}$ of 30 blow per foot (bpf) (which does not liquefy) and the Olson and Stark (2002) residual strength at a $(N_1)_{60cs}$ of 15 bpf. The secant shear modulus was reduced to 5 percent of the maximum shear modulus under the current stress conditions for the residual strength condition.

G.6.3 Constitutive Model Parameters and Distribution

The elastic and Mohr Coulomb/hysteretic and UBCSAND constitutive models were used in the 2D simulation. Given the high strength characteristics of the dense gravels and siltstone and our anticipation that yielding would not occur during dynamic loading, these units were modeled using the elastic/hysteretic model. All other units were modeled with the Mohr-Coulomb/hysteretic model, except for the sands identified by the cone penetrometer test contouring which were modeled with UBCSAND.

As described in the previous sections, several model input parameters are required for each constitutive model. Input parameters such as mass density, friction angle, and permeability were constant over an entire soil unit. Other input parameters including $(N_1)_{60}$, shear wave velocity values, and shear strength were varied with depth and vertical effective stress. Shear wave velocity parameters were increased or decreased based on the change of stress resulting from simulating stockpiling and excavation of soil. The shear strength of the silt units was assigned using the following formula based on the SHANSEP framework to allow for strength increase and decrease from consolidation or unloading:

$$\left(\frac{Su}{P'}\right)_{OC} = \left(\frac{Su}{P'}\right)_{NC} \times OCR^{m}$$

The Su/p' for normally consolidated soils was taken as 0.22 and the superscript "m" = 0.8. Based on in situ strength evaluations, the undrained strength was not allowed to be lower than 500 pounds per square foot (psf). Initial overconsolidation ratios (OCRs) were chosen to approximate the interpreted strength profiles for the existing site topography and site conditions. From the assumed starting OCR, modifications to the geometry such as excavation and stockpile construction were made to the model and solved to equilibrium. It was assumed that in the dynamic loading conditions, the pore pressures changes caused by construction would have dissipated and primary consolidation or unloading of the silt would have completed. Based on the new equilibrium stress states from excavation and stockpiling, the OCR in the model was updated and shear strengths were recalculated and assigned to silt model zones. The result of this procedure is that the strengths beneath the basin generally decreased while the strengths below the stockpile increased relative to their pre-existing conditions.

During dynamic loading, some soils at the site are prone to pore pressure generation and cyclic strength degradation. The loose to medium dense sand layers have the potential to develop excess pore pressures. This behavior was approximated with the constitutive model UBCSAND. Based on CDSS testing and the shear strength characterization in Appendix D, the undrained shear strength of the silts was assigned 75 percent of the static strength to approximate the cyclic strength degradation.

A summary of the FLAC soil input parameters is provided in Table G-3. Soil that has a PI less than 17 is included in layers "L-MD Sands2" and "LowPI Silts2a and b," as shown in Figures G-28 through G-31 and Table G-3.

G.6.4 Dewatered Pore Pressures

An average static groundwater level in the numerical models was set to an elevation equal to +8 feet. Pore pressures were initialized assuming a hydrostatic distribution. For areas where the top of the model (or ground surface) is below an elevation of +8 feet, a normal pressure equal to the weight of the water was applied to the top of the model. For the longitudinal model, a structural element referred to as the "cut-off" wall was used at the gate location to provide a hydraulic barrier that enabled the long-term dewatered pore pressure state of the model to be established. The boundary conditions for both models within the basin at the base of the excavation were modified to represent the zero pressure state imposed by the dewatering system. The model was cycled to provide the steady-state pore pressure distribution in the model.

While establishing initial stress equilibrium, the fluid bulk modulus was set to zero (no pore pressure change calculated) to represent drained conditions. The fluid bulk modulus of groundwater at the site is estimated to be approximately 4.5×10^7 psf. Based on our experience and recommendations in the FLAC user manual, the fluid bulk modulus was set to 1.04×10^7 psf, to represent water in a subsurface environment. In our experience, this methodology substantially decreases model run time; however, it does not result in a significant change in pore pressure generation characteristics and overall results of the model.

To allow for pore pressure changes during the analysis, the fluid-mechanical interaction logic was activated for both models. With this logic, an incremental increase (or decrease) in volumetric strain calculated by the constitutive model represents an incremental decrease (or increase) in the pore volume, which, based on the fluid bulk modulus, causes an incremental increase (or decrease) in the pore pressure. With significant cyclic loading and resulting accumulation of volumetric strains, high excess pore pressures may develop. The dissipation or redistribution of these excess pore pressures can also be modeled by assigning hydraulic conductivity values; however, depending on the relative magnitudes of the hydraulic conductivity, geologic distribution of soil layers and the short duration of loading, the effects of dissipation may be negligible. Based on our experience and the geology of this site, it is our opinion that that the change in pore pressures due to dissipation or redistribution in the short term would not have a significant impact on the results of the model.

G.6.5 Input Ground Motions

The zone sizes of the 2D model varied depending on the stiffness of the material. For the dense gravels and siltstone, zones were approximately 10 feet tall by 10 feet wide. The soils above the dense gravel were modeled with approximate zone sizes ranging from 3 to 6 feet tall from the ground surface to the top of the dense gravel, respectively, and 3 to 5 feet wide. As in the 1D model, FLAC's quiet (compliant) base formulation was applied to the base of the model. Similar to the quiet base formulation, FLAC's free field formulation was applied to the sides of the model to prevent reflection of waves back into the model. The free field formulation simulates the outermost zones of the model and performs calculations in small strain. Due to limitations of the free field formulation's ability to model user-defined constitutive models, the built-in Mohr-Coulomb and hysteretic damping models were used on the outermost zones.

The dynamic loading of both models was implemented in the same manner. A shear stress time history, required when modeling a compliant base, was applied to the bottom of the model. The input horizontal shear stress time history was calculated from the outcrop velocity time histories of the spectrally matched ground motion time histories using the following equation:

$$\tau_{XY} = -2 \times \frac{1}{2} \times \frac{\gamma_{TOT}}{g} \times Vs \times v_c$$

where:

τ_{XY} = Horizontal shear stress
γ_{TOT} = Total density
g = Gravity constant
Vs = Shear wave velocity of the medium
v_c = Velocity time history

G.6.6 Soil-Structure Interaction

The 2D transverse numerical models included evaluation of the crane trestle structure, basin slope, basin slab, and toe walls at the bottom of the slopes and piles. The crane trestle structure consisted of two longitudinal beams and a transverse beam. The longitudinal beams form the upper walls and pile cap for the trestle piling. The transverse beam was connected to the two longitudinal beams at the top of the piles. The connection between the transverse and longitudinal beams, and the longitudinal beam and trestle piles, was made very stiff to essentially allow for full transfer of moments. The base slab and toe walls were modeled with structural beam elements and connected with a fixed connection. The basin piles were connected to the basin slab with a pin connection with zero tensile capacity. All piles and the basin slab were modeled with a linear elastic and, in some cases (gantry piles, basin piles, and slab), purely plastic moment curvature behavior. A plastic moment parameter was used which allows plastic hinging of the piles and basin slab to occur whenever the plastic moment is reached. Plastic yield moments were assigned for the pile cap connections on the crane trestle, along the length of both the crane trestle and basin piles, and on the base slab. All structural parameters including plastic yield moments, stiffness, and areas were provided by the structural engineer. An additional mass was included in the longitudinal beams to represent a portion of the ballast mass to be included in the dynamic evaluation.

The structural pile elements interact with the soil grid through normal and shear springs. Strengths and stiffness were assigned to the shear springs to approximate skin friction along the pile, except for the bottom-most shear spring, which was assigned strength and stiffness parameters to approximate end bearing. The normal springs were assigned strength and stiffness properties to approximate the limit state at which the soil would begin to flow around the piles. These parameters were determined by evaluating a series of 2D horizontal slice simulations in which piles of varying size and spacing were "pushed" through a horizontal slice of soil. The strength parameters for the normal springs were assigned frictional angles that are dependent on the effective stress acting normal to the pile. This methodology allows for the strength of the normal springs to vary as pore pressures change in the various soil units. The resulting soil structure interaction spring and structural parameters are summarized in Table G-5.

The trestle and toe walls and the basin slab were modeled with structural beam elements that interacted with the soil grid through interfaces. The interfaces were assigned a friction angle equal to 0.75 times the tangent of the internal friction angle of the adjacent soils and stiffness parameters roughly 10 times the stiffness of the adjacent soils, based on recommendations in the FLAC user manuals. The interfaces allow sliding and gapping between the soil and structures to occur.

G.6.7 Results

G.6.7.1 Longitudinal

The primary objective for the longitudinal models was to assess the free-field soil movements at the location of the gate structure. Results of horizontal displacements at the gate along the centerline of the basin are shown in Figure 16 of the main report. The average horizontal displacement is approximately 1.0 foot and is moving to the north into the basin. The direction of movement is based on shear stress development around and below the sheet pile cutoff in the direction of the basin. Although the basin surface is relatively flat, the shear stress develops towards the basin direction because of an imbalanced pore pressure on either side of the sheet pile wall. Cyclic shear stress pulses from the dynamic loading result in an increase of the pore pressures and reduction of the shear strength in the sandy zones. Once the shear strength in the sandy zones dropped below the existing shear stress demand, the soil strained toward the basin.

Results of horizontal displacements in the longitudinal direction on the outside of the basin are shown in Figure 17. In this case, the horizontal displacement is approximately 1 to 6 feet with an average of 3 feet in the direction of the Chehalis River at the crest of the river bank. The lateral displacement magnitudes are of the same approximate magnitude to the estimates provided in the Request for Proposals based on Youd and others (2003) empirical correlations. The direction of movement is based on shear stress around and below the sheet pile cutoff developed toward the Chehalis River because of the surface geometry including the nearby slope bank. Contour plots from a representative ground motion of horizontal displacement and excess pore pressure ratio are shown in Figures G-40 and G-41.

Free field-horizontal displacements along the gate structure based on limited longitudinal simulations between the scenarios described above are shown in Figure 18. This plot shows that at the base of the basin slopes the horizontal displacements begin to shift from moving into the basin to moving toward the Chehalis River. The transition is abrupt and is primarily related to the higher ground level on the north side of the sheet pile cut-off and reduction of the imbalanced pore pressures.

G.6.7.2 Transverse

The primary objective of the transverse simulations was to assess the impacts of dynamic soil movements on the crane trestle structure. Horizontal soil displacements, horizontal pile displacements, pile moments, and pile node angular displacements at the end of shaking and maximum pile shear stresses are presented in Figures 20 through 29 for cross sections along the northern and southern portions of the basin. Contour plots from a representative ground motion of horizontal displacement and excess pore pressure ratio are shown in Figures G-42 and G-43. Note that angular displacements represent the rotation of the pile nodes caused by bending forces and should not be misinterpreted as pile curvature. The structural response results shown were evaluated by the structural engineer, KPFF Consulting Engineers, against structural performance criteria.

G.7 REFERENCES

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TABLE G-1
CONSIDERED TIME HISTORIES

Earthquake Event	Recording Station	Component Direction
Central Chile (1985)	Valparaiso U.F.S.M	70°
Central Chile (1985)	Valparaiso U.F.S.M	160°
Central Chile (1985)	Valparaiso El Almendral	70°
Central Chile (1985)	Valparaiso El Almendral	160°
Central Chile (1985)	Villita, Mexico	90°
Central Chile (1985)	Villita, Mexico	360°
Central Chile (1985)	Rapel	North-South
Peru (2007)	Parcona	East-West
Peru (2007)	Parcona	North-South
Peru (2007)	ICA	East-West
Peru (2007)	ICA	North-South
Lima, Peru (1974)	Callao, Lima, Peru	North-South
Lima, Peru (1974)	Callao, Lima, Peru	East-West
Lima, Peru (1974)	La Molina, Lima, Peru	Longitudinal
Lima, Peru (1974)	La Molina, Lima, Peru	Transverse
Michoacan, Mexico (1985)	Zihuatanejo, Mexico	90°
Michoacan, Mexico (1985)	Zihuatanejo, Mexico	360°
Michoacan, Mexico (1985)	La Union, Mexico	90°
Michoacan, Mexico (1985)	La Union, Mexico	360°
Miyagi-Ken-Oki, Japan (1978)	TH019	N41E
Miyagi-Ken-Oki, Japan (1978)	TH019	E41S
Tokachi-Oki, Japan (1968)	TH029	East-West
Tokachi-Oki, Japan (1968)	TH029	North-South

TABLE G-2
REFERENCE TIME HISTORIES

Earthquake Event	Recording Station	Magnitude	Distance (km)	Component Direction
Central Chile (1985)	Valparaiso U.F.S.M.	7.8	93	70°
Michoacan, Mexico (1985)	Zihuatanejo, Mexico	8.1	132	90°
Michoacan, Mexico (1985)	Zihuatanejo, Mexico	8.1	132	360°
Michoacan, Mexico (1985)	La Union, Mexico	8.1	83	90°
Michoacan, Mexico (1985)	La Union, Mexico	8.1	83	360°
Tokachi-Oki, Japan (1968)	TH029	8.3	71	East-West
Tokachi-Oki, Japan (1968)	TH029	8.3	71	North-South

TABLE G-3
FLAC INPUT PARAMETERS - LONGITUDINAL MODEL

Soil Unit[1]	Elevation[3] (ft)	Constitutive Soil Model	Hysteretic Damping & Modulus Reduction Curve	Total Unit Weight (pcf)	Friction Angle (deg)	Dilation (deg)	Cohesion (psf)	Clean Sand SPT Blow Count, $N_{1,60,CS}$	Shear Modulus Gmax (ksf)	Bulk Modulus Bmax (ksf)
Fill	15 to 2	Mohr-Coulomb w/ Hysteretic	Vucetic & Dobry (PI=30)	105	30	0	100		189 to 201	568 to 604
Base Fill	-9 to -11	Mohr-Coulomb w/ Hysteretic	Gravel (Rollins)	135	37	5	0		496 to 496	455 to 455
Silts	6 to -22	Mohr-Coulomb w/ Hysteretic	Vucetic & Dobry (PI=30)	105	0	0	Su/p = 0.22		159 to 176	477 to 529
L-MD Sands[2]	-11 to -22	UBCSAND		120	33	1.5	0	15	438 to 448	584 to 597
Silts2	-14 to -67	Mohr-Coulomb w/ Hysteretic	Vucetic & Dobry (PI=30)	105	0	0	Su/p = 0.22		182 to 557	544 to 1670
L-MD Sands2[2]	-35 to -76	UBCSAND		125	33	1	0	10	709 to 1275	946 to 1701
Silts3	-45 to -105	Mohr-Coulomb w/ Hysteretic	Vucetic & Dobry (PI=30)	105	0	0	Su/p = 0.22		485 to 801	1455 to 2402
Dense Sands	-89 to -110	Mohr-Coulomb w/ Hysteretic	EPRI Sand (51ft to 120ft)	130	37	5	0		1411 to 3510	1545 to 3844
Gravels	-105 to -185	Elastic w/ Hysteretic	Gravel (Rollins)	135					3731 to 8209	3420 to 7525
Siltstone	-185 to -225	Elastic w/ Hysteretic	EPRI Rock (250ft to 500ft)	140					11980 to 25940	10980 to 23780

Notes:
1. See Figures G-28 and G-29 for geometry of soil units.
2. Silts with a PI <=17 were modeled with the L-MD Sands soil unit.
3. Elevations represent the extents of soil units. Some sloped units may have a large extent but a small thickness.
4. Blank values indicate parameters that are not applicable to the applied constitutive model.

TABLE G-4
FLAC INPUT PARAMETERS - TRANSVERSE MODEL

Soil Unit[1]	Elevation[5] (ft)	Constitutive Soil Model	Hysteretic Damping & Modulus Reduction Curve	Total Unit Weight (pcf)	Friction Angle (deg)	Dilation (deg)	Cohesion (psf)[2]	$N_{1,60,CS}$	Shear Modulus Gmax (ksf)	Bulk Modulus Bmax (ksf)
RipRap	13 to -7	Mohr-Coulomb w/ Hysteretic	Gravel (Rollins)	145	43	7	0		1320	1210
Stockpile	38 to 18	Mohr-Coulomb w/ Hysteretic	Vucetic & Dobry (PI=30)	90	18	0	0		244 to 417	732 to 1252
Fill	18 to 4	Mohr-Coulomb w/ Hysteretic	Vucetic & Dobry (PI=30)	95	30	0	100		173 to 186	518 to 557
Base Fill	11 to -12	Mohr-Coulomb w/ Hysteretic	Gravel (Rollins)	135	37	5	0		1313	1203
Silts	9 to -11	Mohr-Coulomb w/ Hysteretic	Vucetic & Dobry (PI=30)	95	0	0	500 to 800 (level ground) 700 to 920 (stockpile)		207 to 231	621 to 694
LowPI Silts	-35 to -75	Mohr-Coulomb w/ Hysteretic	Vucetic & Dobry (PI=15)	115	0	0	960 to 2225 (basin) 1350 to 2750 (level ground) 1500 to 2925 (stockpile)		319 to 860	691 to 1864
L-MD Sands	-6 to -20	UBCSAND		120	33	1 to 2	0	9 to 20	408 to 431	374 to 395
Silts2	-16 to -35	Mohr-Coulomb w/ Hysteretic	Vucetic & Dobry (PI=30)	110	0	0	500 to 535 (basin) 710 to 850 (level ground) 980 to 1090 (stockpile)		276 to 398	828 to 1193
L-MD Sands2	-50 to -60	UBCSAND		125	33	1.5	0	15	998 to 1072	915 to 983
Silts3	-75 to -90	Mohr-Coulomb w/ Hysteretic	Vucetic & Dobry (PI=30)	110	0	0	1385 to 1570 (basin) 1580 to 1750 (level ground) 1675 to 1835 (stockpile)		487 to 771	1462 to 2314
Dense Sands	-90 to -105	Mohr-Coulomb w/ Hysteretic	EPRI Sand (51ft to 120ft)	130	37	5	0		1460 to 2676	1599 to 2931
Gravels	-105 to -185	Elastic w/ Hysteretic	Gravel (Rollins)	135					3634 to 9433	3331 to 8647
Siltstone	-185 to -225	Elastic w/ Hysteretic	EPRI Rock (250ft to 500ft)	140					12218 to 29947	11200 to 27452

Notes:
1. See Figures G-28 and G-29 for geometry of soil units.

2. Cohesion ranges are based the increases and decreases of effective stress and OCR from excavation of the basin and stockpile construction and a 25% reduction to account for cyclic degradation in the silts with a PI <= 17 (LowPI Silts).
3. Shear wave velocities represent model conditions in static equilibrium that are based on stress changes from excavation and stockpiling of soil.
4. Blank values indicate parameters that are not applicable to the applied constitutive model.
5. Elevations represent the extents of soil units. Some sloped units may have a large extent but a small thickness.

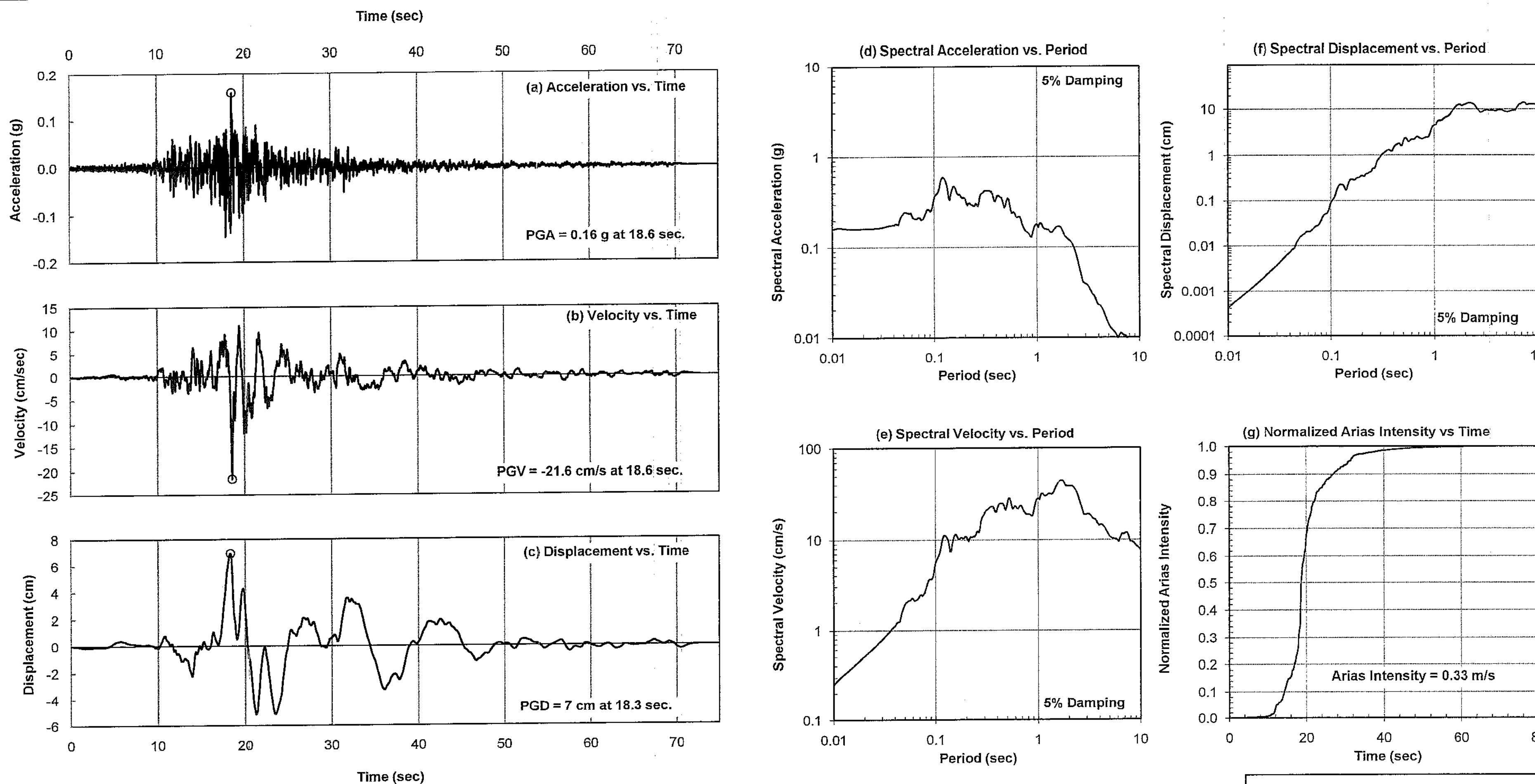
TABLE G-5a
FLAC SSI INPUT PARAMETERS - TRANSVERSE MODEL

			SSI Spring Parameters					Structural Pile Parameters							
Pile Type	Soil Unit	Property Number	Coupling Spring Normal Interface Friction for piles, nfric (deg)	Coupling-Spring Shear Friction Angle for pile, sfric (deg)	Adhesion (psf)	Shear Stiffness (psf)	Normal Stiffness (psf)	Composite Mass Density (scf)	Young's Modulus (ksf)	Diameter (in)	Perimeter (ft)	Spacing (ft)	Area (ft²)	Moment of Inertia (ft⁴)	Plastic Moment (lb-ft)
Gantry Pile (24")	Fill	3001	81.6	23.4	0	3102	33333	30.81	4176000	24	6.28	20	0.58	0.134	971667
	Silts	3002	76.2	0.0	2963	35558	34800	30.81	4176000	24	6.28	20	0.58	0.134	971667
	LowPI Silts2	3003	74.4	0.0	5027	60319	83067	30.81	4176000	24	6.28	20	0.58	0.134	971667
	L-MD Sands	3004	80.9	26.0	0	14322	71367	30.81	4176000	24	6.28	20	0.58	0.134	971667
	Silts2	3005	68.4	0.0	3506	42072	50600	30.81	4176000	24	6.28	20	0.58	0.134	971667
	L-MD Sands2	3006	80.9	26.0	0	35254	175300	30.81	4176000	24	6.28	20	0.58	0.134	971667
	Silts3	3007	70.9	0.0	3031	36372	104333	30.81	4176000	24	6.28	20	0.58	0.134	971667
	Dense Sands	3008	81.7	29.5	0	64132	309667	30.81	4176000	24	6.28	20	0.58	0.134	971667
	Gravels	3009	81.7	34.0	0	76630	309667	30.81	4176000	24	6.28	20	0.58	0.134	971667
	Gravel (End Bearing)	3010	81.7	0.0	157080	942478	309667	30.81	4176000	24	6.28	20	0.58	0.134	971667
Basin Pile (18")	Fill	3011	81.6	23.4	0	2326	25000	15.22	4176000	18	4.71	17.5	0.14	0.039	485000
	Silts	3012	76.2	0.0	2222	26668	26100	15.22	4176000	18	4.71	17.5	0.14	0.039	485000
	LowPI Silts2	3013	74.4	0.0	3770	45239	62300	15.22	4176000	18	4.71	17.5	0.14	0.039	485000
	L-MD Sands	3014	80.9	26.0	0	10742	53525	15.22	4176000	18	4.71	17.5	0.14	0.039	485000
	Silts2	3015	68.4	0.0	2630	31554	37950	15.22	4176000	18	4.71	17.5	0.14	0.039	485000
	L-MD Sands2	3016	80.9	26.0	0	26441	131475	15.22	4176000	18	4.71	17.5	0.14	0.039	485000
	Silts3	3017	70.9	0.0	2273	27279	78250	15.22	4176000	18	4.71	17.5	0.14	0.039	485000
	Dense Sands	3018	81.7	29.5	0	48099	232250	15.22	4176000	18	4.71	17.5	0.14	0.039	485000
	Gravels	3019	81.7	34.0	0	57472	232250	15.22	4176000	18	4.71	17.5	0.14	0.039	485000
	Gravel (End Bearing)	3020	81.7	0.0	176715	706858	232250	15.22	4176000	18	4.71	17.5	0.14	0.039	485000

TABLE G-5b
FLAC STRUCTURAL PARAMETERS - TRANSVERSE MODEL

			Structural Pile Parameters					
Pile Type	Soil Unit	Property Number	Mass Density (scf)	Young's Modulus (ksf)	Spacing (ft)	Area (ft²)	Moment of Inertia (ft⁴)	Plastic Moment (lb-ft)
Gantry Pile (24")	Longitudinal Beam	1001	3.79	524160	1	4.31	1.73	
	Transverse Beam	1002	4.97	524160	20	7.56	4.77	
	Toe Wall	1003	4.97	524160	1	1.00	0.08	
	Base Slab	1004	4.97	524160	1	1.50	0.28	20000
	Basin Pile-Slab Connection	1010	15.22[1]	4176000	17.5	0.14	0.04	
	Transverse-Longitudinal Connection	1020	4.97	524160	1	7.56	1.73	
	Longitudinal-Pile Connection	1030	4.97	524160	20	2.94	0.37	766083

Zihuatanejo 090 REF.xlsm Printed: 1/10/2011 5:45 PM



NOTES:

1. This figure is the AZIH090 reference ground motion.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

REFERENCE GROUND MOTION
1985, MICHOACAN, MEXICO
ZIHUATANEJO, 90 DEGREES

August 2010 21-1-21190-015

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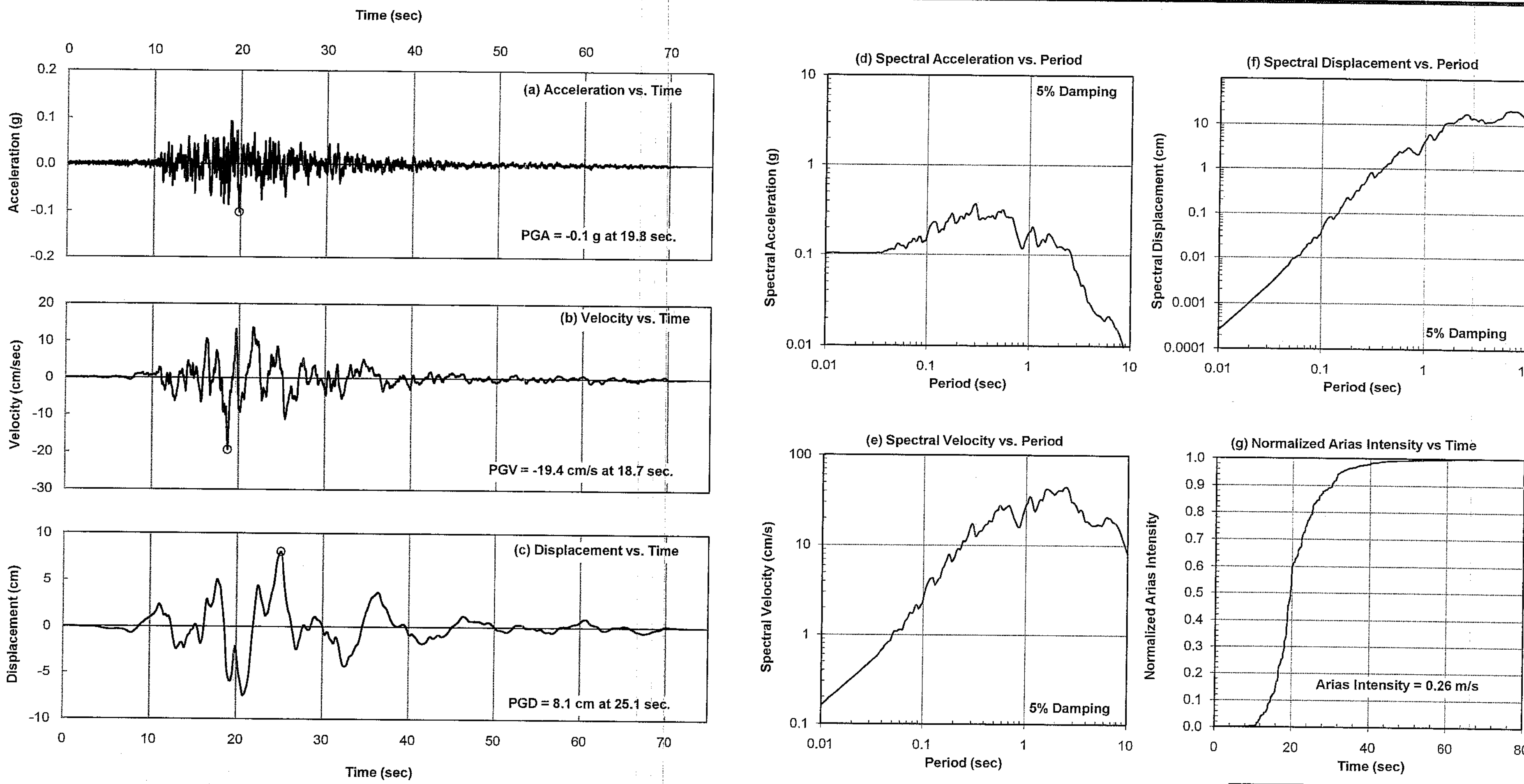
FIG. G-1

Zihuatanejo 360 REF.xlsm Printed: 1/10/2011 5:46 PM



NOTES:

1. This figure is the AZIH360 reference ground motion.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

REFERENCE GROUND MOTION
1985, MICHOACAN, MEXICO
ZIHUATANEJO, 360 DEGREES

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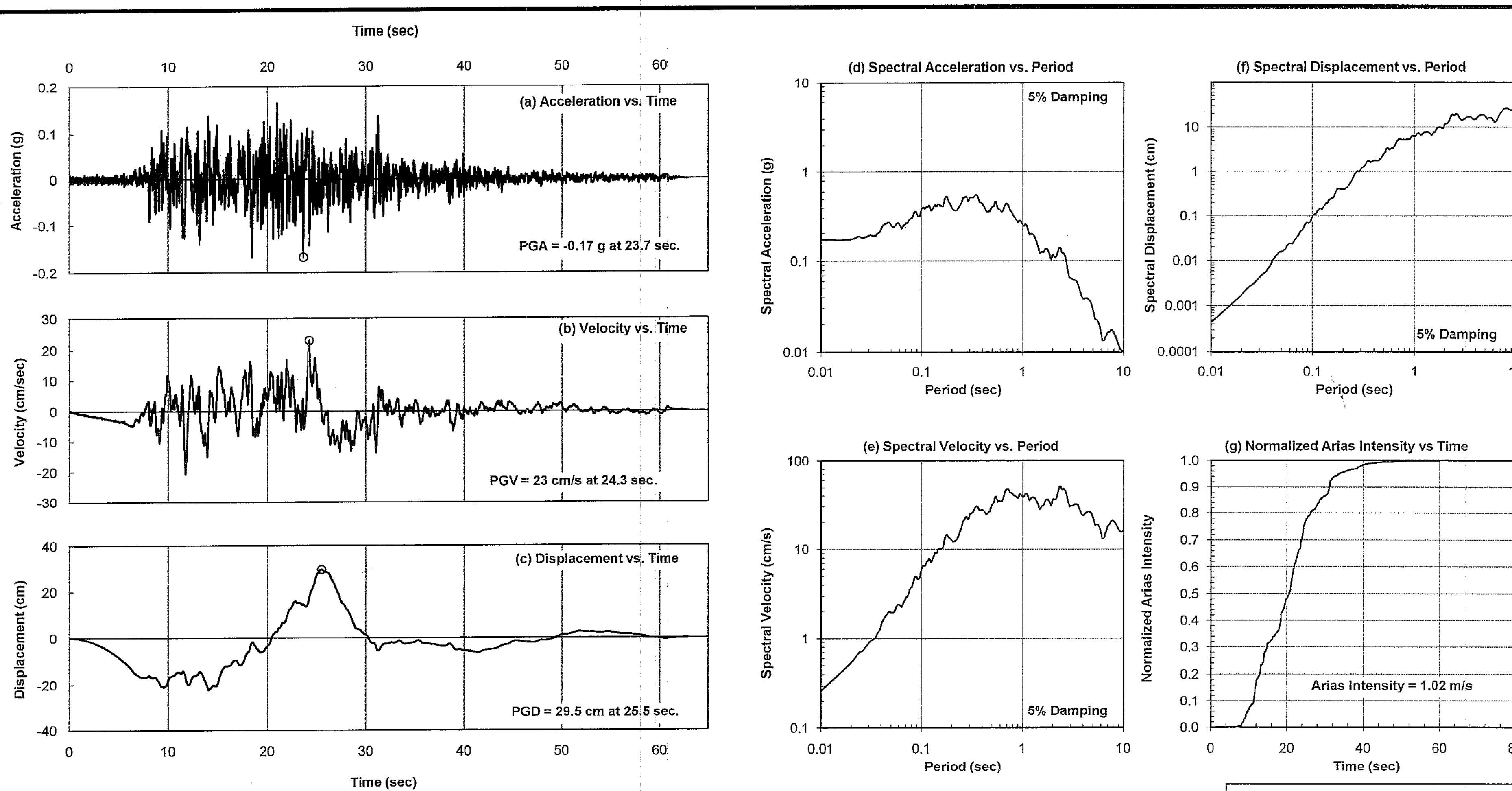
FIG. G-2

La Union 360 REF.xlsm Printed: 1/10/2011 5:38 PM



NOTES:

1. This figure is the MICN00 reference ground motion.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

REFERENCE GROUND MOTION
1985, MICHOACAN, MEXICO
LA UNION, 360 DEGREES

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FIG. G-3

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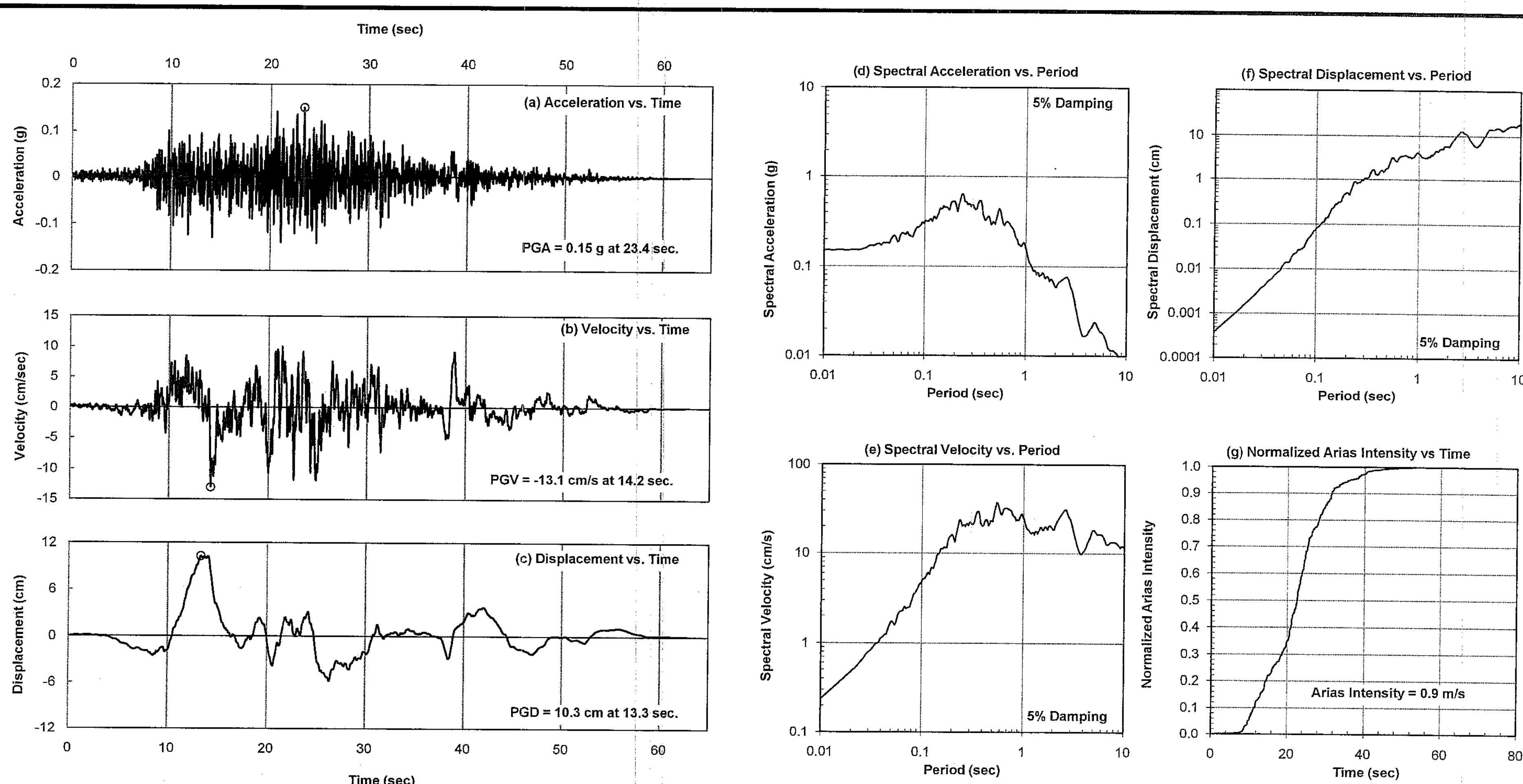

NOTES:

1. This figure is the MICN90 reference ground motion.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

REFERENCE GROUND MOTION
1985, MICHOACAN, MEXICO
LA UNION, 90 DEGREES

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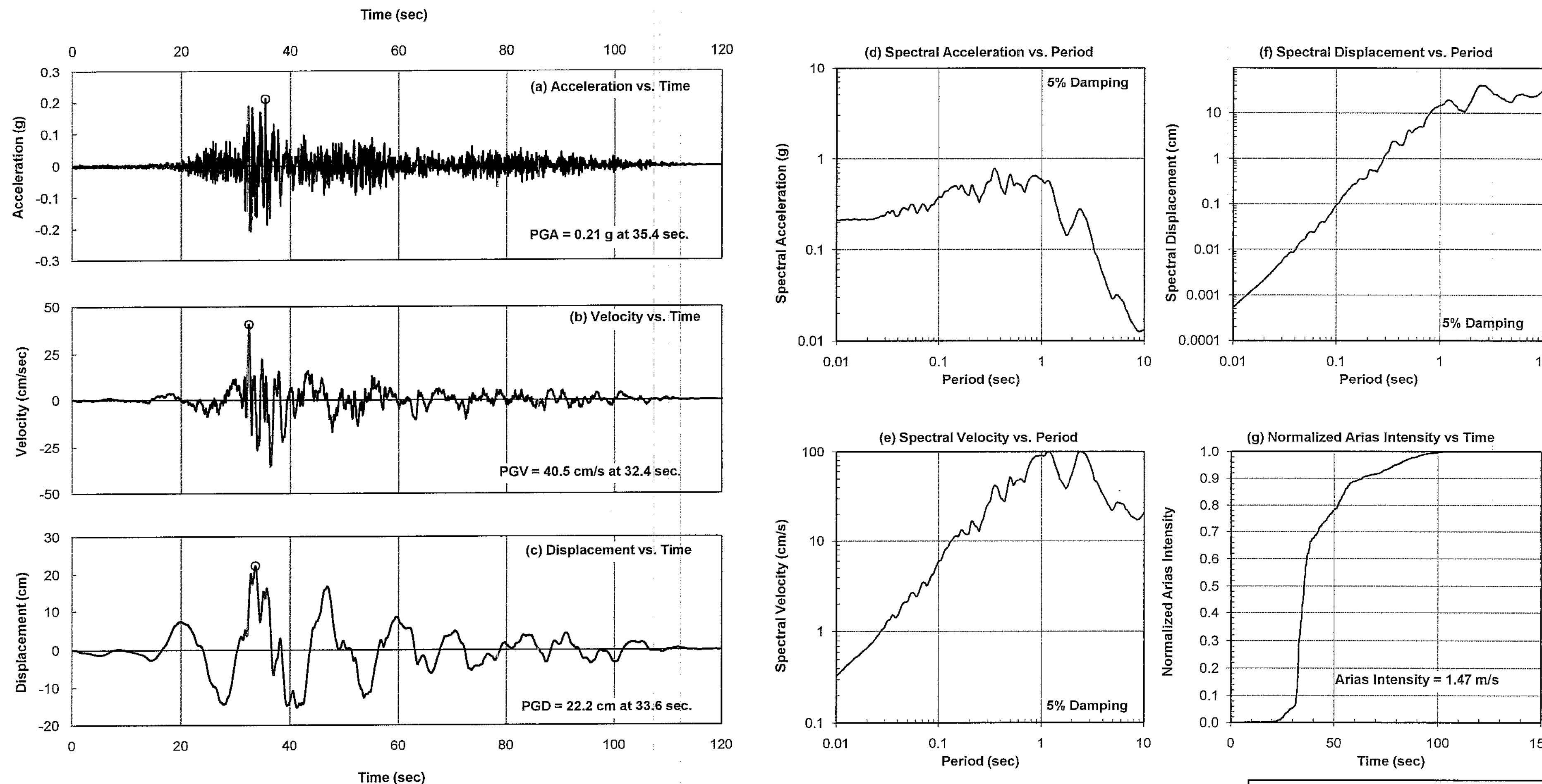
FIG. G-4

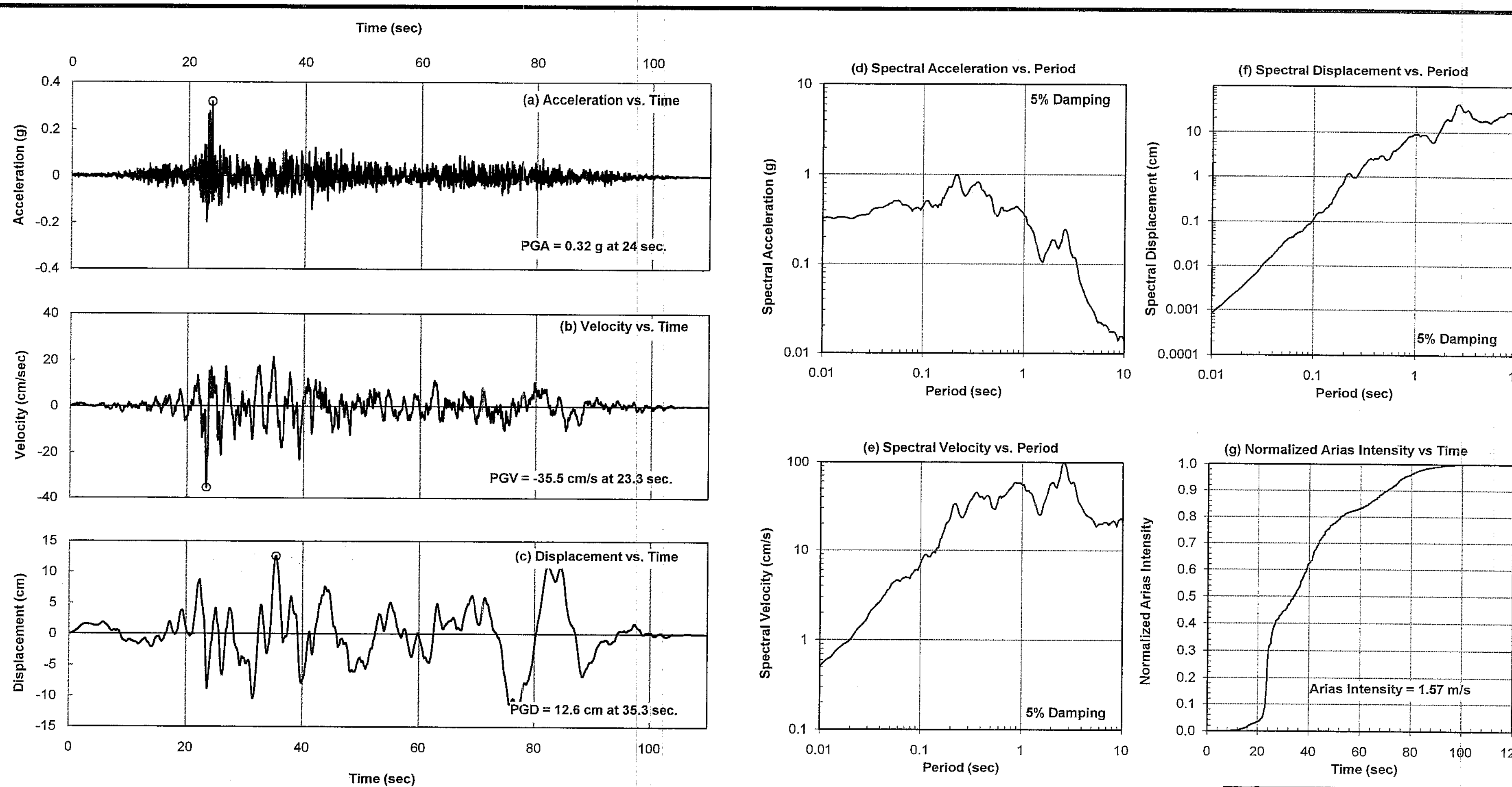


NOTES:

1. This figure is the TOKEW reference ground motion.

SR 520 Pontoon Casting Facility Aberdeen, Washington	
REFERENCE GROUND MOTION 1968, TOKACHI-OKI, JAPAN TH029, 90 DEGREES	
August 2010	21-1-21190-015
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. G-5

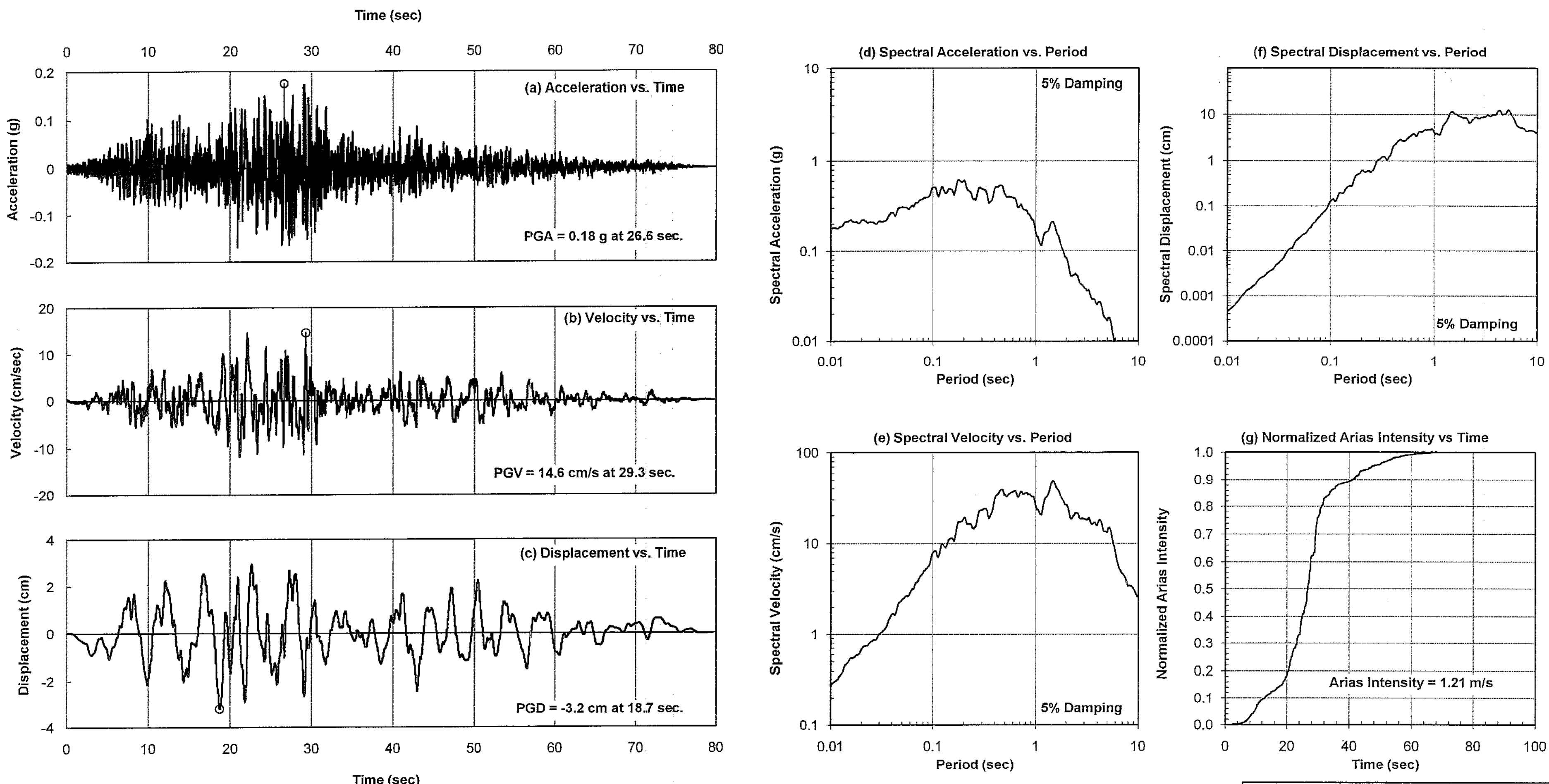
Tokachi NS REF.xlsm Printed: 1/10/2011 5:42 PM



NOTES:

1. This figure is the TOKNS reference ground motion.

SR 520 Pontoon Casting Facility Aberdeen, Washington	
REFERENCE GROUND MOTION 1968, TOKACHI-OKI, JAPAN TH029, 360 DEGREES	
August 2010	21-1-21190-015
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. G-6**



NOTES:

1. This figure is the UFSM70 reference ground motion.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

REFERENCE GROUND MOTION
1985, CENTRAL CHILE
VALPARAISO UFSM, 70 DEGREES

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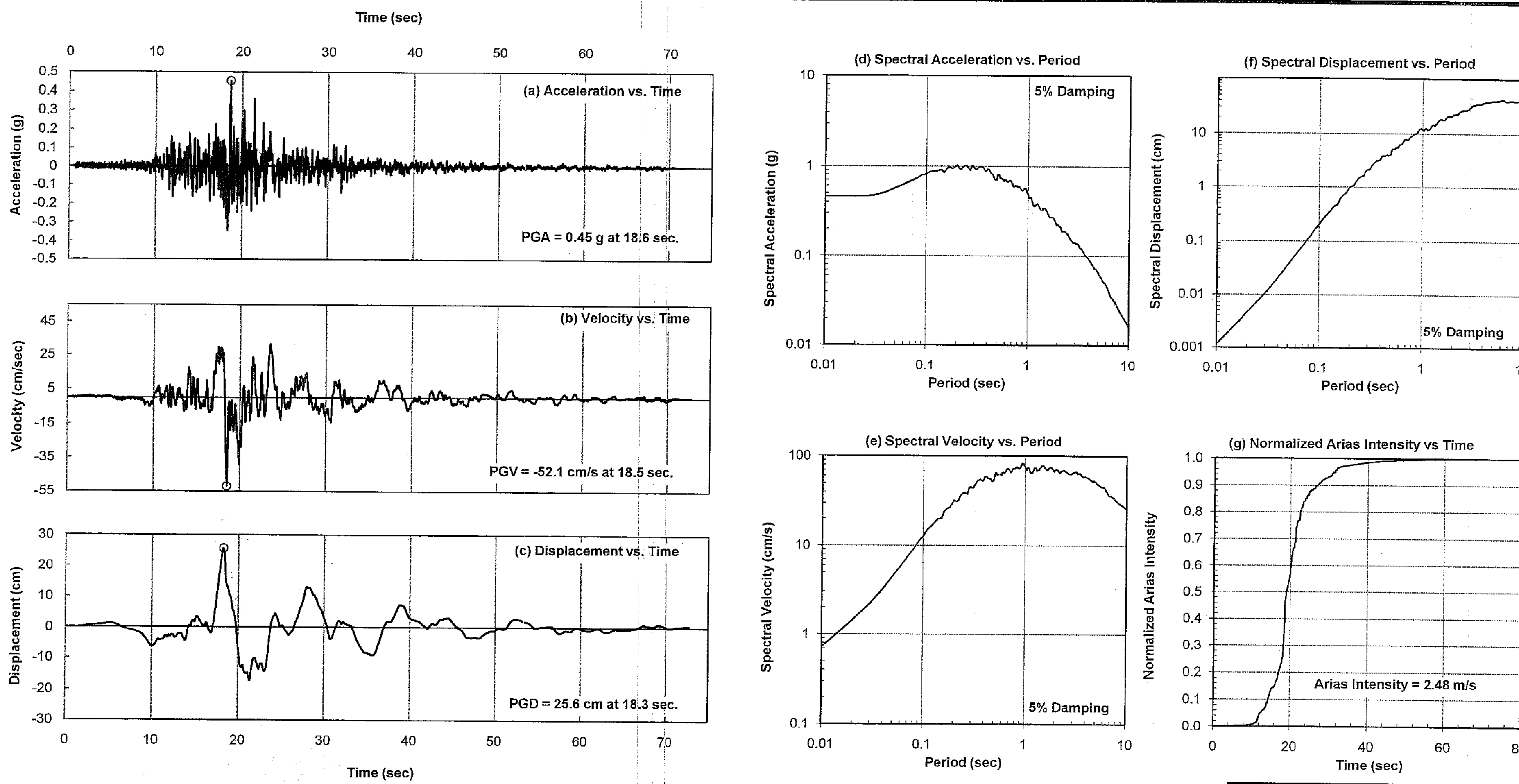
FIG. G-7



NOTES:

1. This figure is the AZIH090 matched ground motion.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

MATCHED GROUND MOTION
1985, MICHOACAN, MEXICO
ZIHUATANEJO, 90 DEGREES

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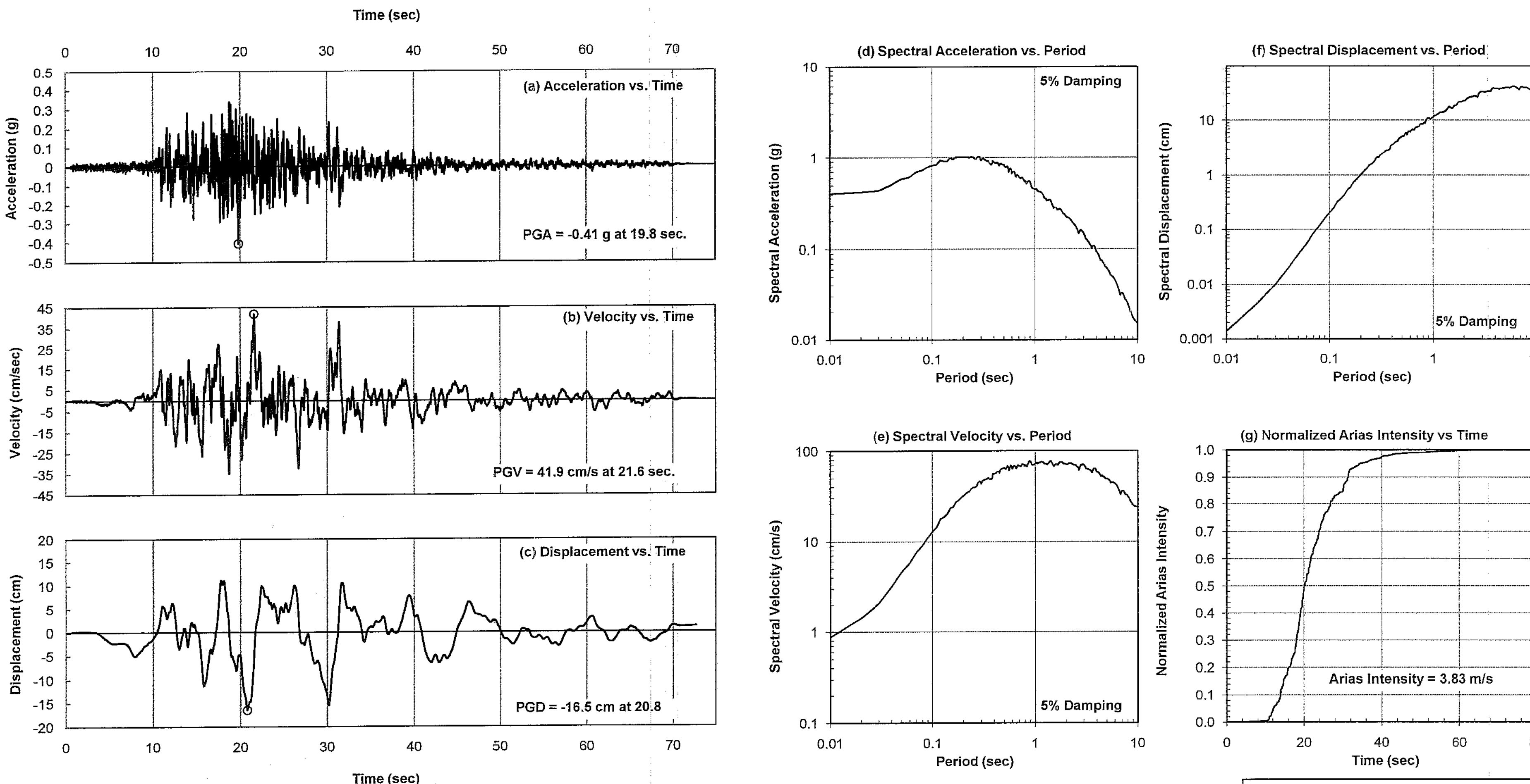
FIG. G-8

azih360c_r22a_FINAL.xlsm Printed: 8/24/2010 2:23 PM



NOTES:

1. This figure is the AZIH360 matched ground motion.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

MATCHED GROUND MOTION
1985, MICHOACAN, MEXICO
ZIHUATANEJO, 0 DEGREES

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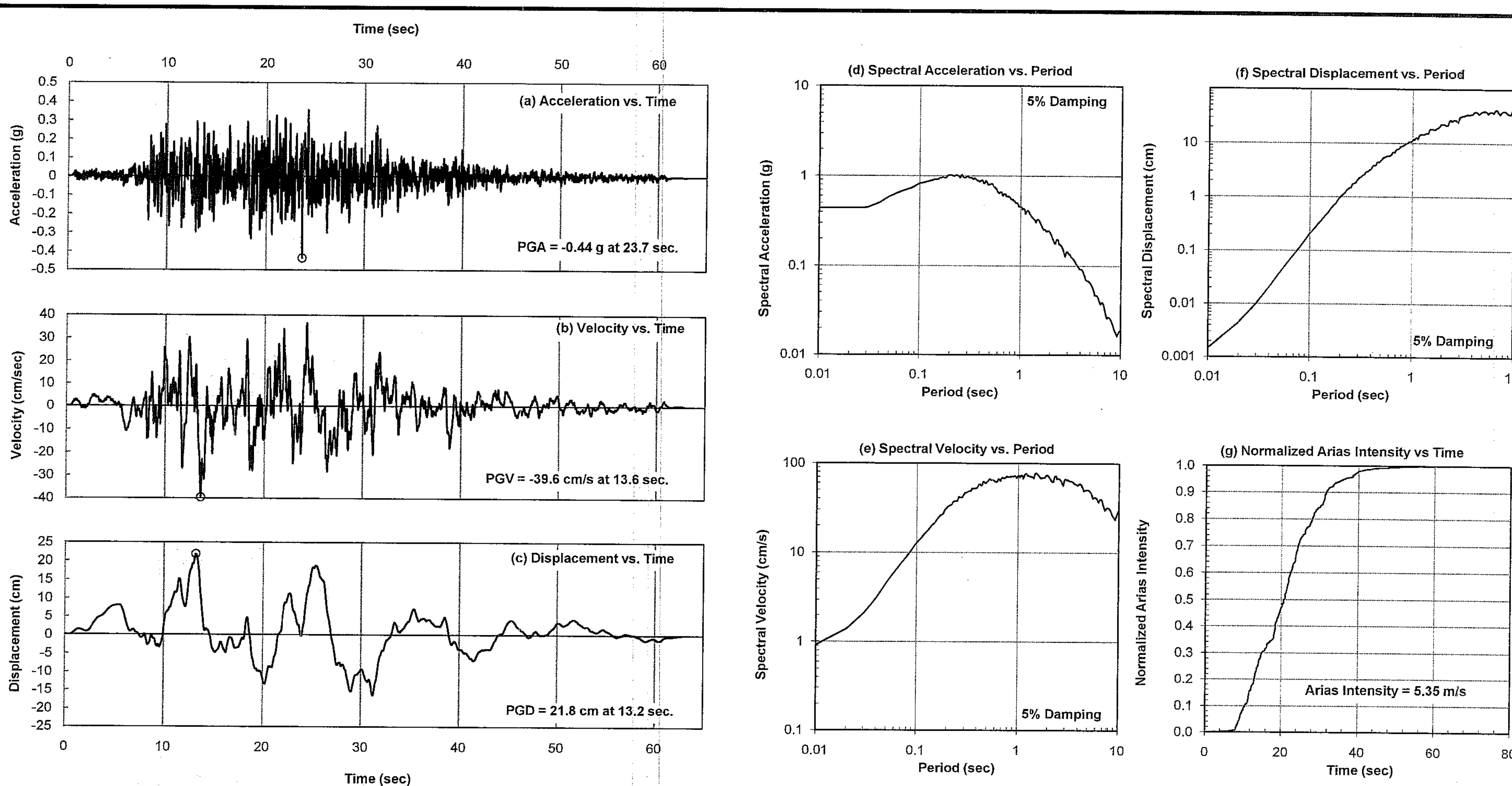
FIG. G-9



NOTES:

1. This figure is the MICN00 matched ground motion.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

MATCHED GROUND MOTION
1985, MICHOACAN, MEXICO
LA UNION, 0 DEGREES

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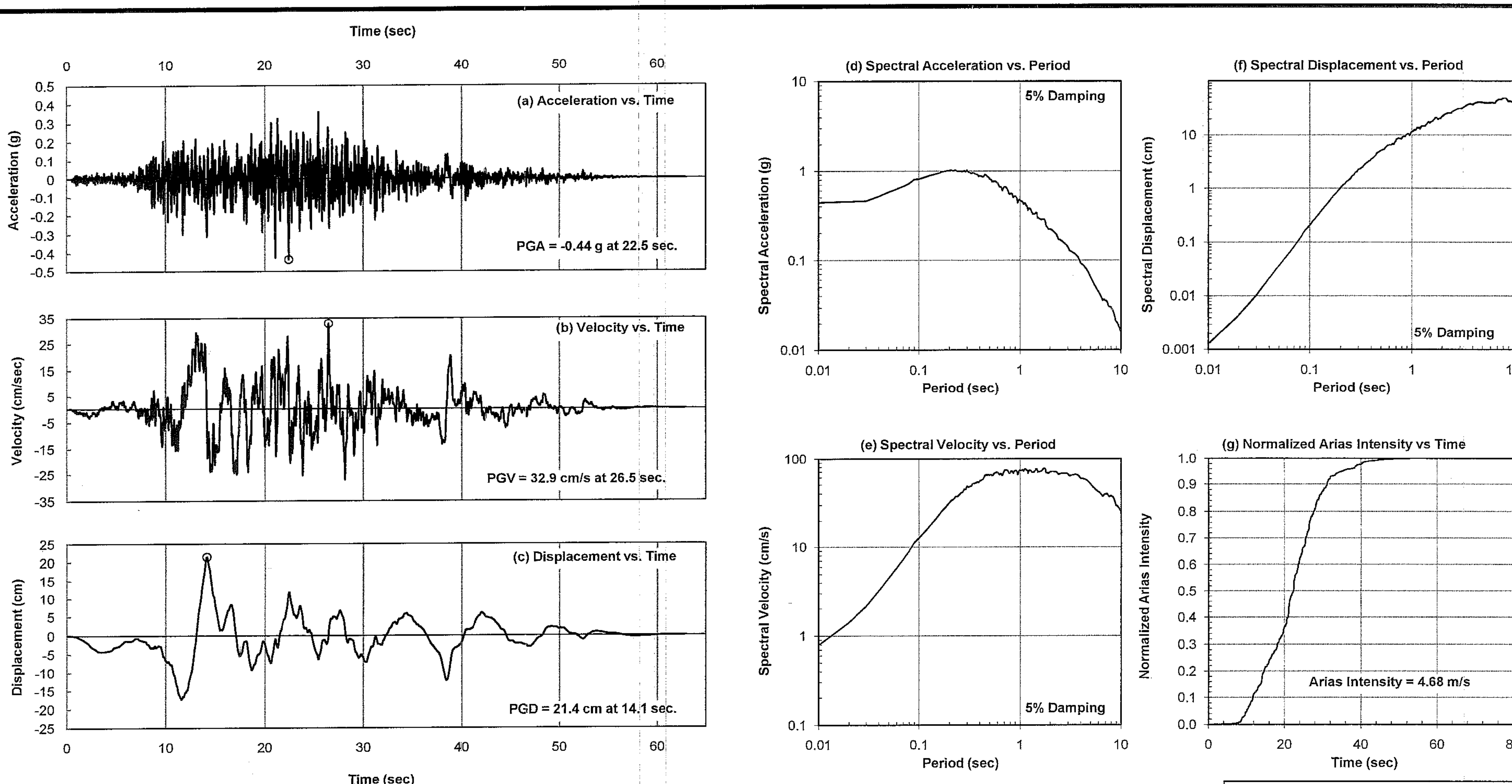
FIG. G-10



NOTES:

1. This figure is the MICN90 matched ground motion.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

MATCHED GROUND MOTION
1985, MICHOACAN, MEXICO
LA UNION, 270 DEGREES

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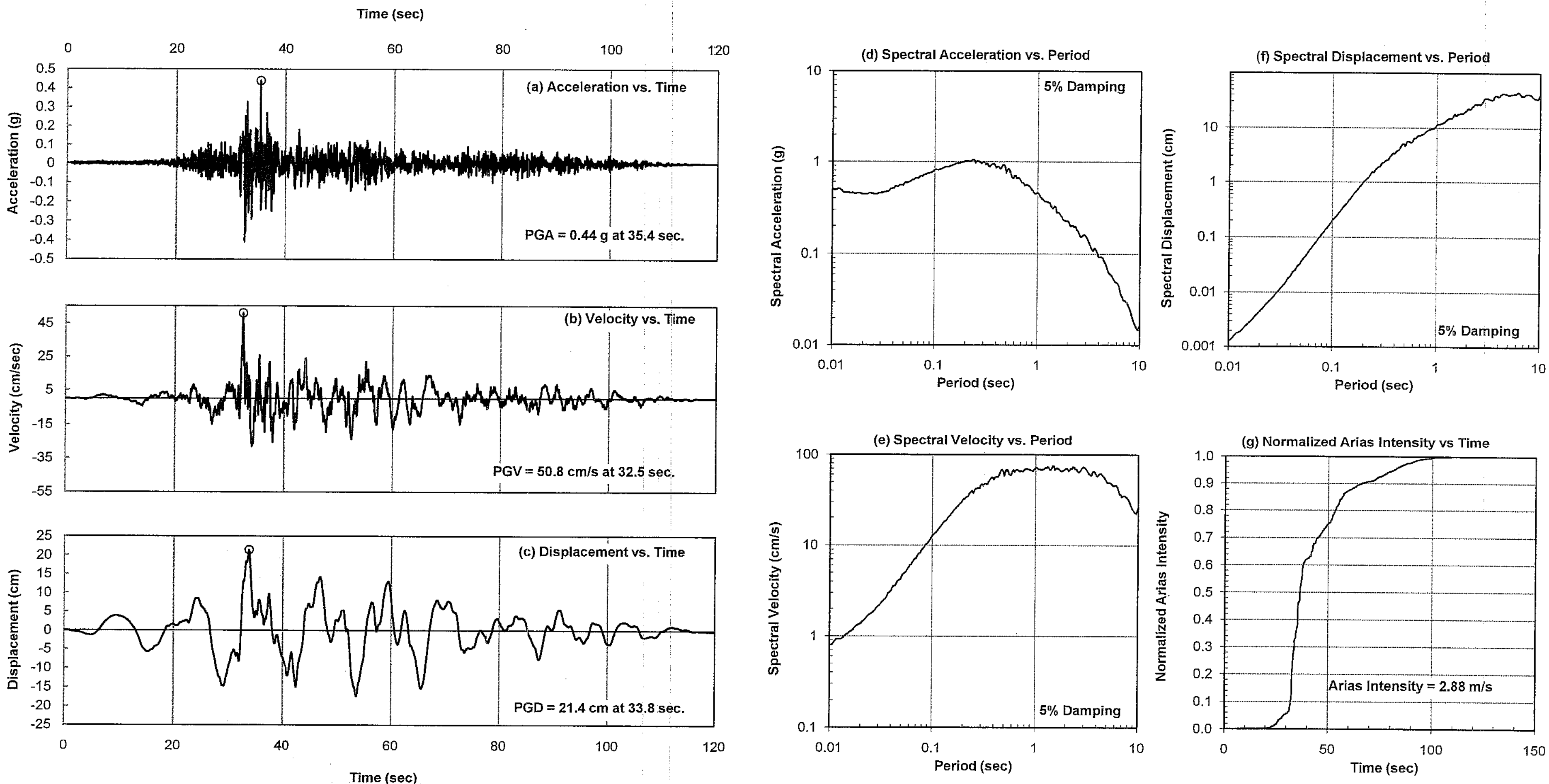
FIG. G-11

TokachiEW_r03a_FINA.xism Printed: 1/10/2011 5:28 PM



NOTES:

1. This figure is the TOKEW matched ground motion.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

MATCHED GROUND MOTION
1968, TOKACHI-OKI, JAPAN
TH029, 90 DEGREES

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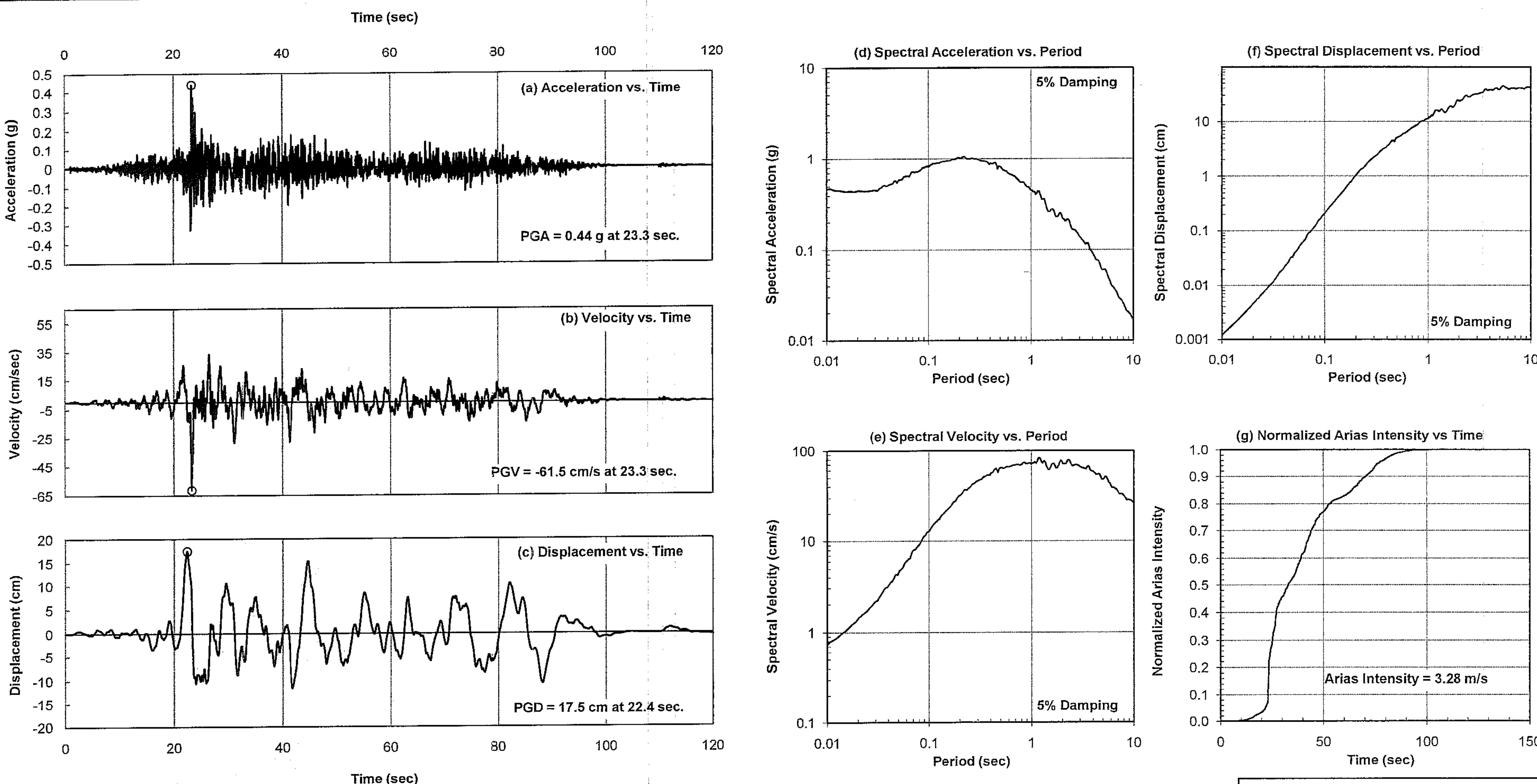
FIG. G-12

TokachiNS_r03a_FINAL.xlsm Printed: 1/10/2011 5:32 PM



NOTES:

1. This figure is the TOKNS matched ground motion.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

MATCHED GROUND MOTION
1968, TOKACHI-OKI, JAPAN
TH029, 0 DEGREES

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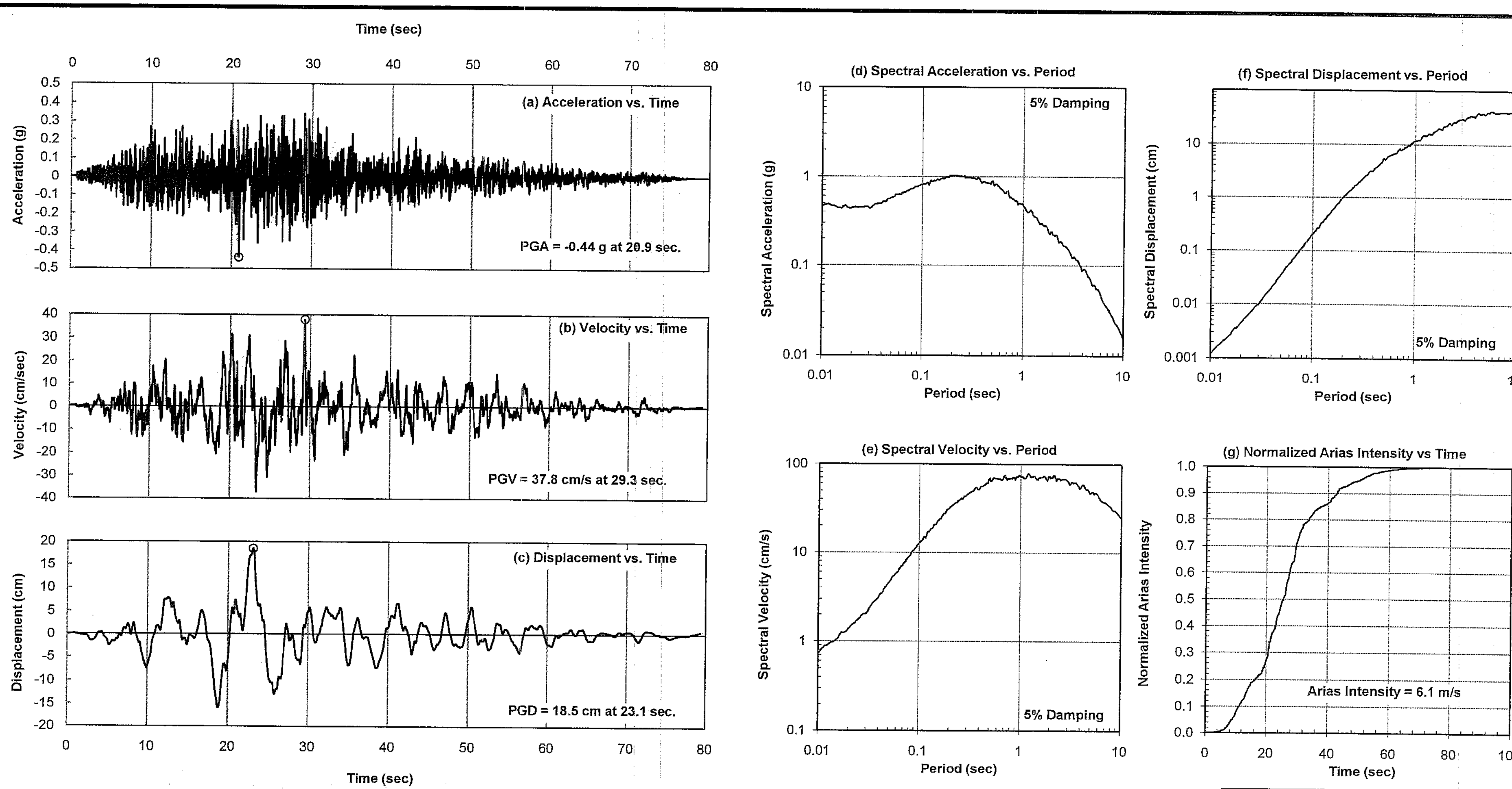
FIG. G-13



NOTES:

1. This figure is the UFSM70 matched ground motion.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

MATCHED GROUND MOTION
1985, VALPARAISO, CHILE
UFSM, 70 DEGREES

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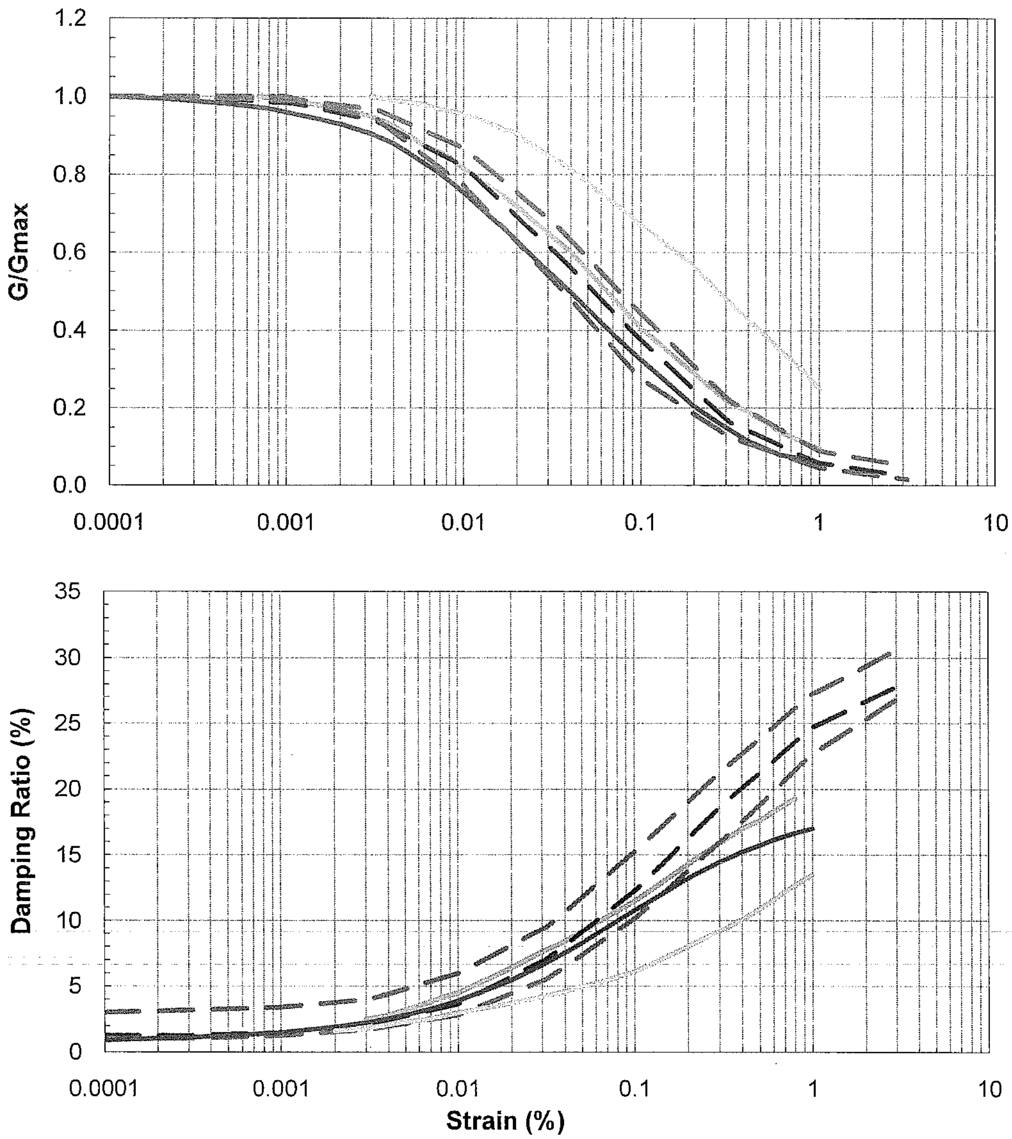
FIG. G-14



MODULUS DEGRADATION	DAMPING
Vucetic & Dobry (1991) PI=15 OCR=1 to 15	Vucetic & Dobry (1991) PI=15 OCR=1 to 8
EPRI (1993) Soil 21-50 feet	EPRI (1993) Soil 21-50 feet
EPRI (1993) Soil 51-120 feet	EPRI (1993) Soil 51-120 feet
EPRI (1993) Rock 251-500 feet	EPRI (1993) Rock 251-500 feet
Rollins et al. (1998) Mean Gravel	Rollins et al. (1998) Mean Gravel

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MODULUS DEGRADATION AND DAMPING CURVES

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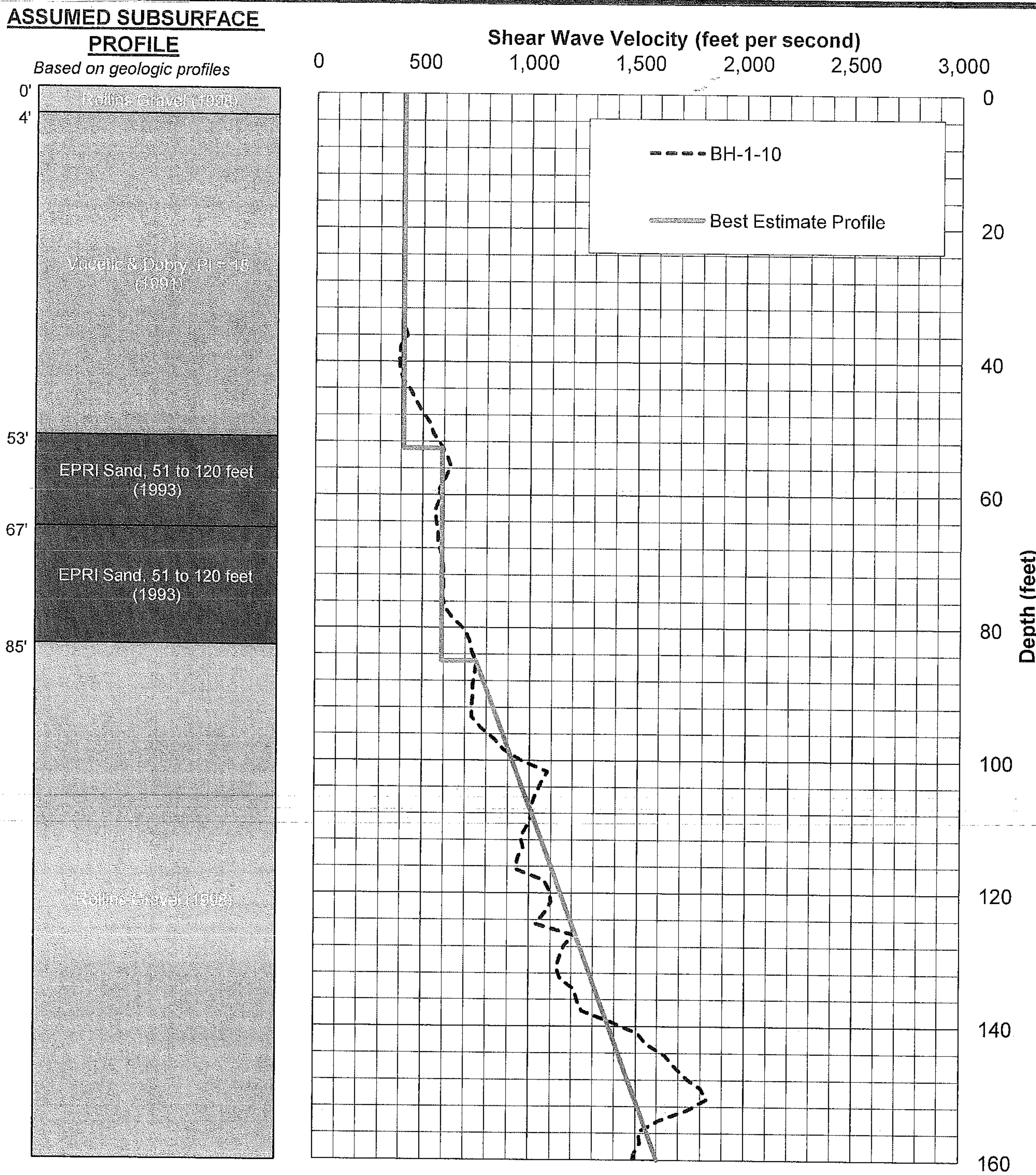
FIG. G-15

BH-1-10.xlsm 8/24/2010

jnb



NOTES

1. The shear wave velocity profile is based on measurements from Boring BH-1-10.
2 We selected the soil unit weight and modulus and damping curves based on the subsurface conditions encountered in the borings, and our judgment.
3 pcf = pounds per cubic foot; PI = plasticity index

SR 520 Pontoon Casting Facility
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SHEAR WAVE VELOCITY PROFILE
BORING BH-1-10

August 2010 | 21-1-21190-015

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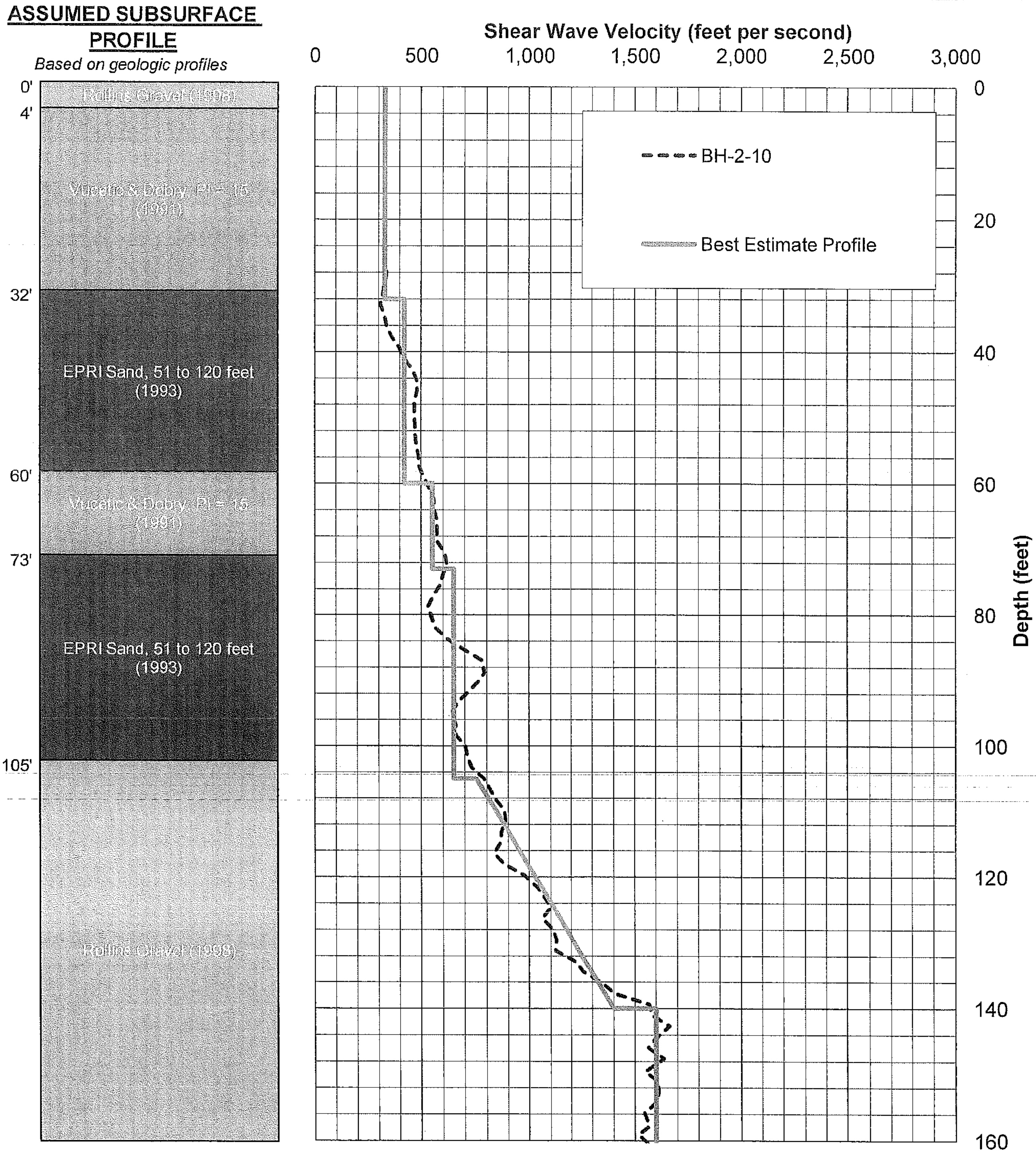
FIG. G-16



NOTES

1. The shear wave velocity profile is based on measurements from Boring BH-2-10.
2. We selected the soil unit weight and modulus and damping curves based on the subsurface conditions encountered in the borings, and our judgment.
3. pcf = pounds per cubic foot; PI = plasticity index

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SHEAR WAVE VELOCITY PROFILE
BORING BH-2-10

August 2010 21-1-21190-015

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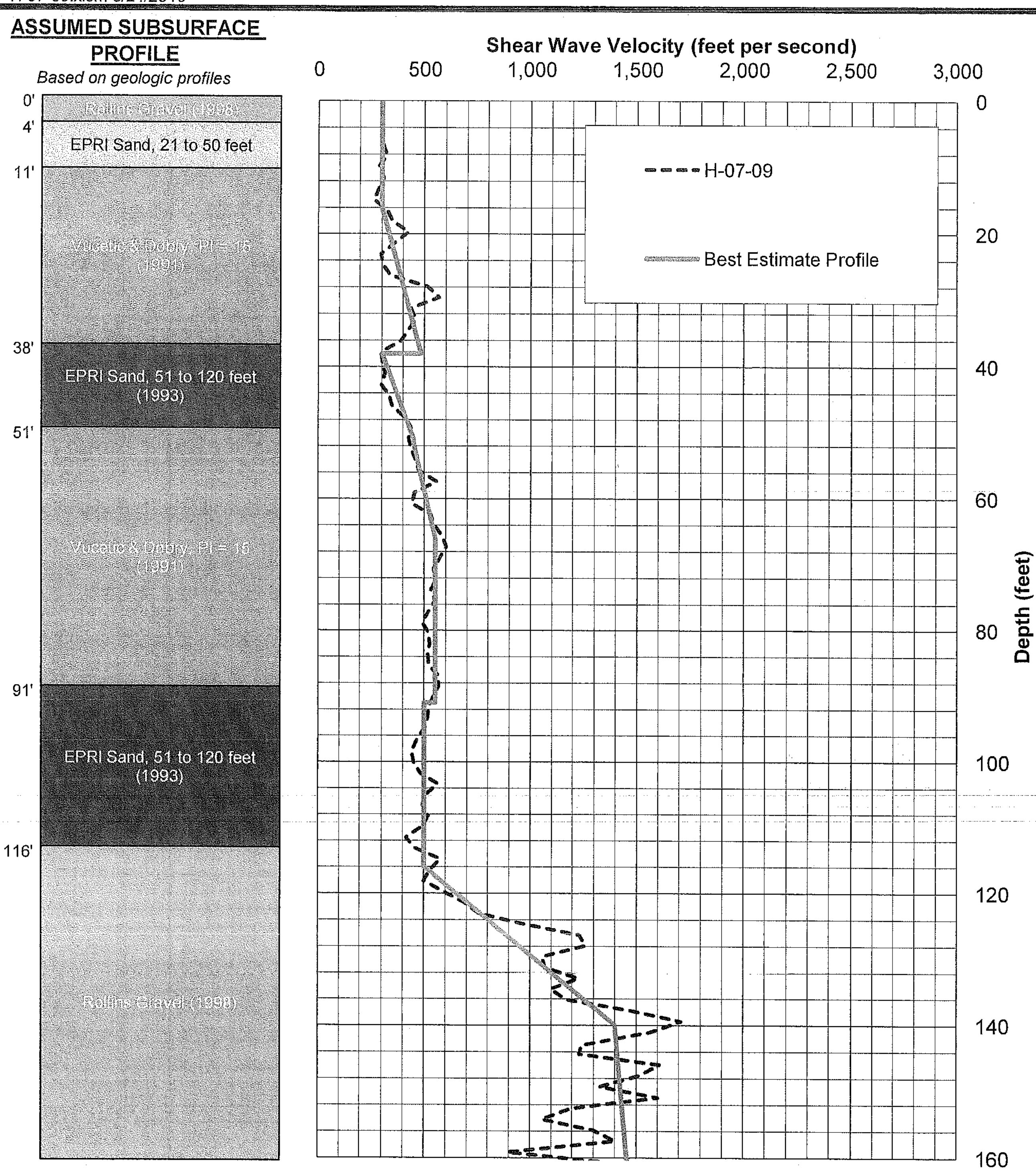
FIG. G-17



NOTES

1. The shear wave velocity profile is based on measurements from Boring H-07-09.
2. We selected the soil unit weight and modulus and damping curves based on the subsurface conditions encountered in the borings, and our judgment.
3. pcf = pounds per cubic foot; PI = plasticity index

SR 520 Pontoon Casting Facility
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SHEAR WAVE VELOCITY PROFILE
BORING H-07-09

August 2010 21-1-21190-015

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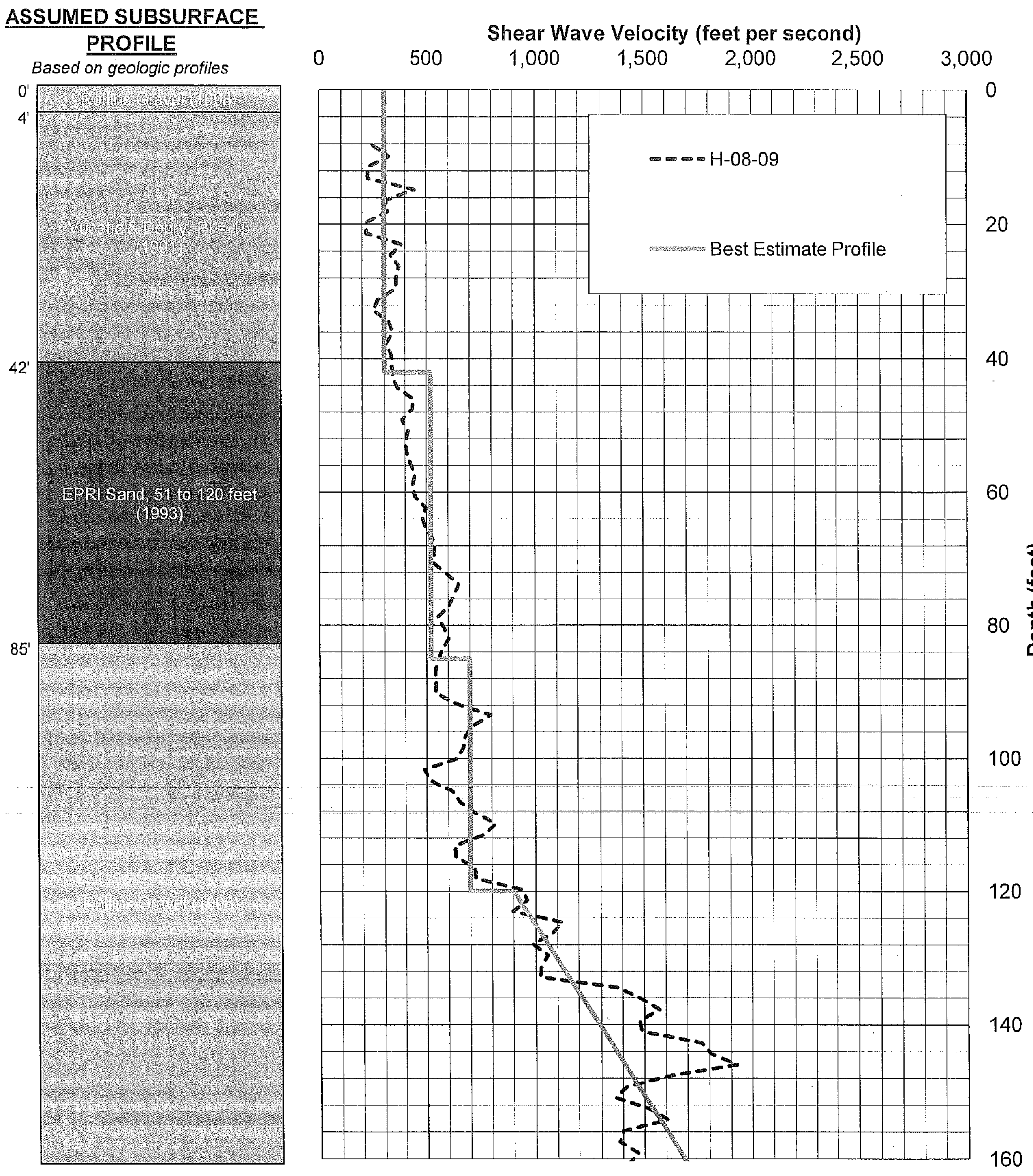
FIG. G-18



NOTES

1. The shear wave velocity profile is based on measurements from Boring H-08-09.
2. We selected the soil unit weight and modulus and damping curves based on the subsurface conditions encountered in the borings, and our judgment.
3. pcf = pounds per cubic foot; PI = plasticity index

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SHEAR WAVE VELOCITY PROFILE
BORING H-08-09

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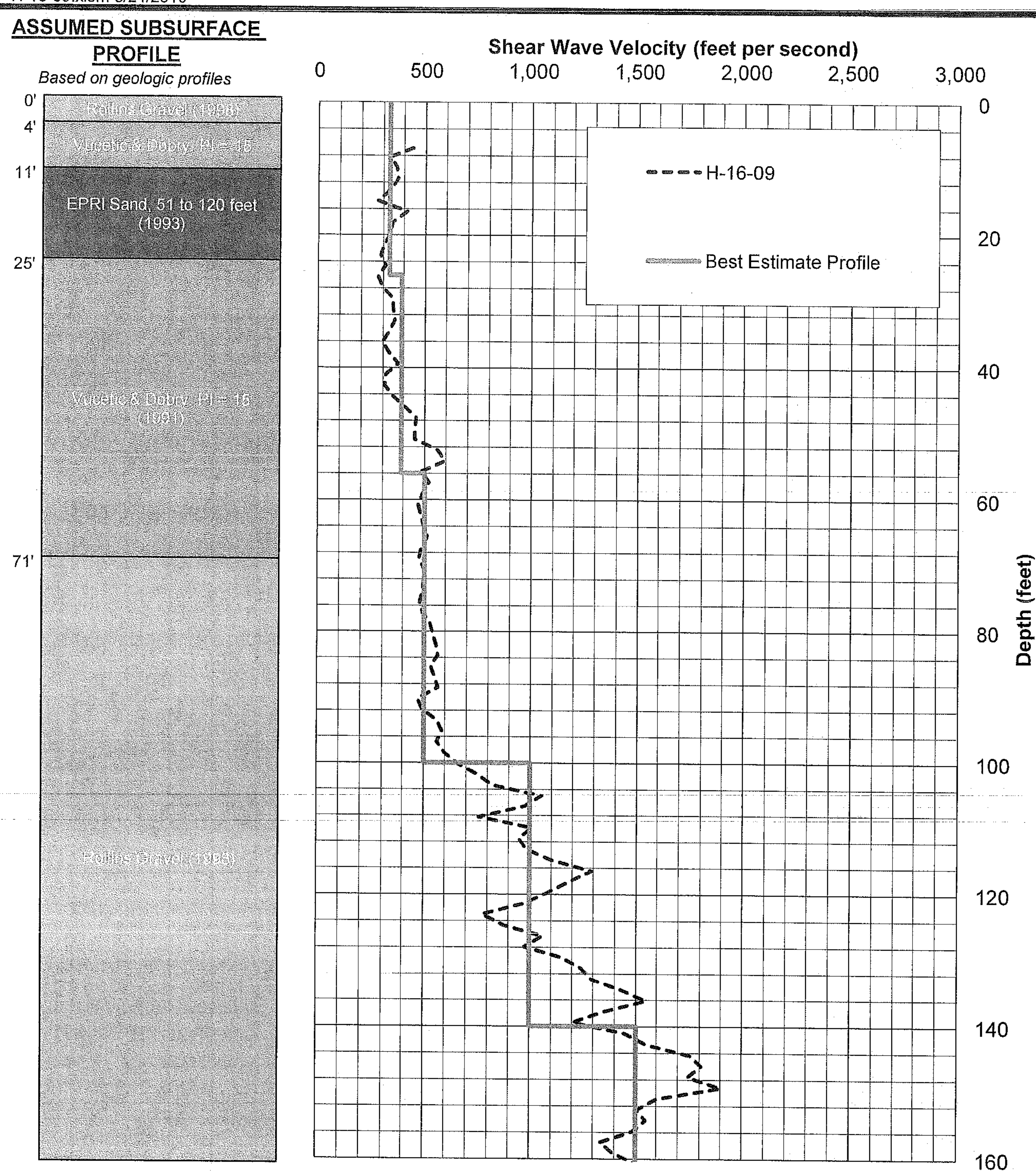
FIG. G-19

H-16-09.xlsm 8/24/2010

jnb



NOTES

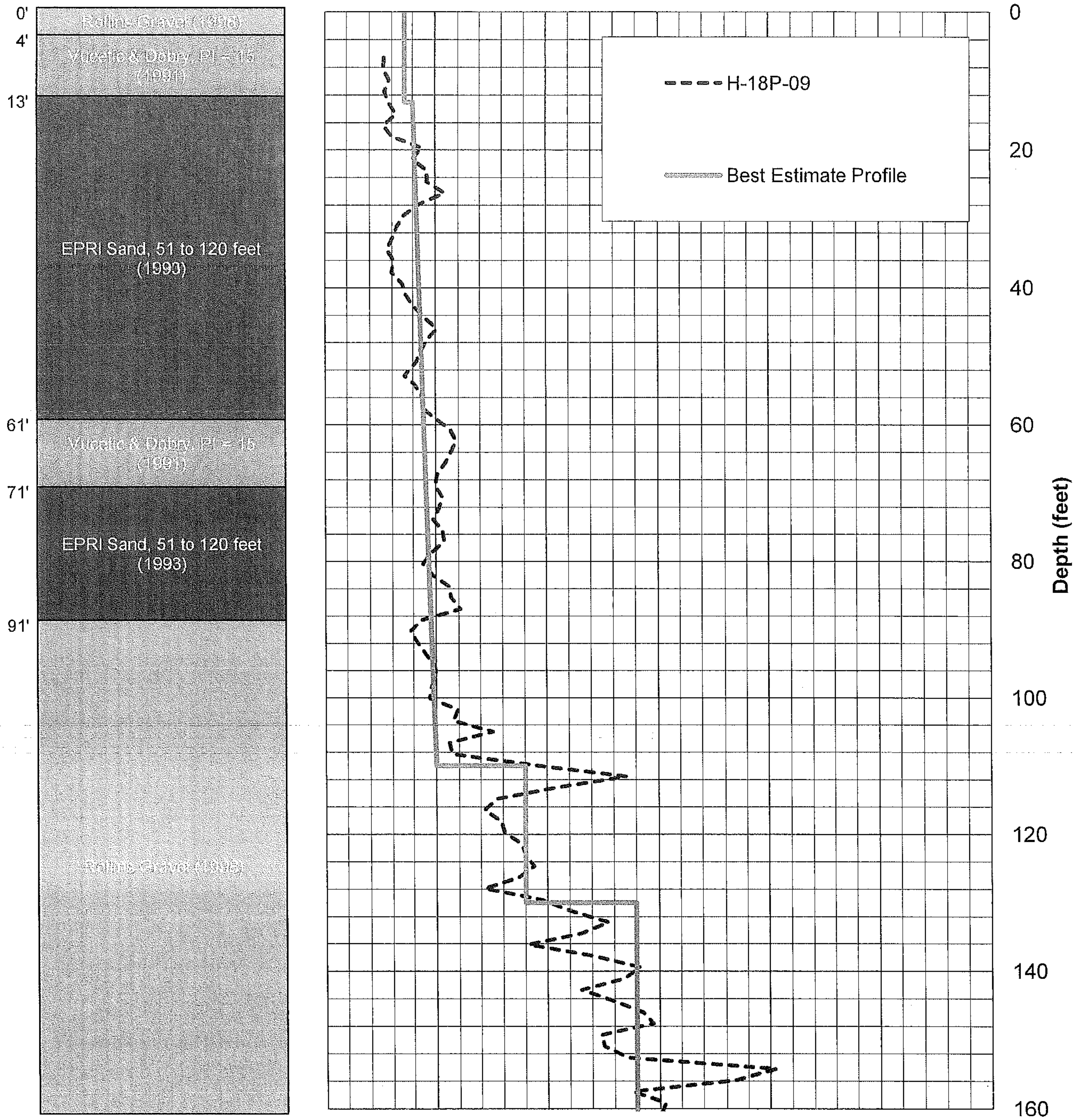
1. The shear wave velocity profile is based on measurements from Boring H-16-09.
2. We selected the soil unit weight and modulus and damping curves based on the subsurface conditions encountered in the borings, and our judgment.
3. pcf = pounds per cubic foot; PI = plasticity index

SR 520 Pontoon Casting Facility Aberdeen, Washington	
SHEAR WAVE VELOCITY PROFILE BORING H-16-09	
August 2010	21-1-21190-015
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. G-20**

ASSUMED SUBSURFACE PROFILE

Based on geologic profiles

Shear Wave Velocity (feet per second)



NOTES

1. The shear wave velocity profile is based on measurements from Boring H-18P-09.
2. We selected the soil unit weight and modulus and damping curves based on the subsurface conditions encountered in the borings, and our judgment.
3. pcf = pounds per cubic foot; PI = plasticity index

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SHEAR WAVE VELOCITY PROFILE
BORING H-18P-09

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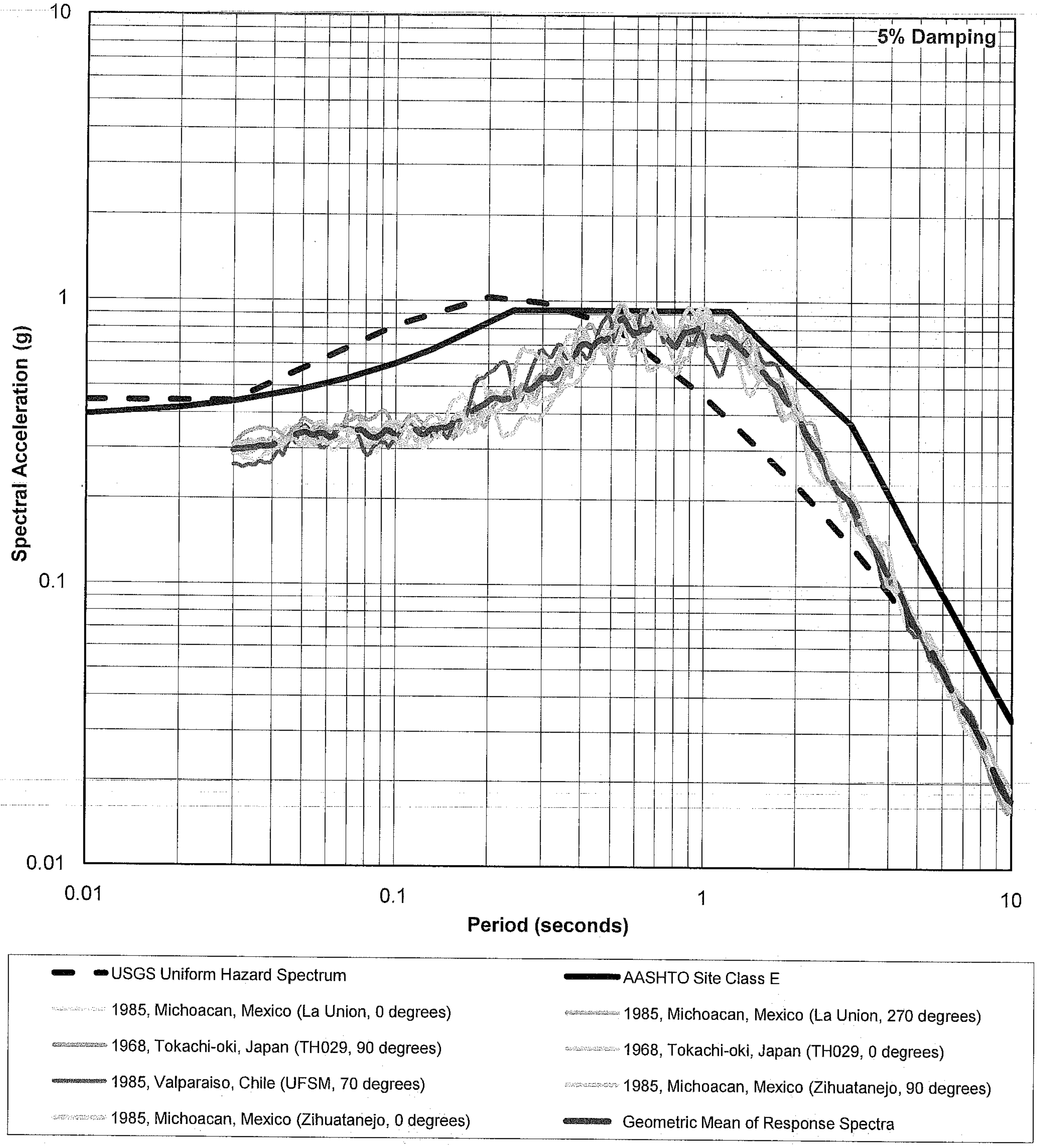
FIG. G-21

FLAC 1DSetupv3.23 (Tot Stress).xlsm 8/24/2010



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ONE-DIMENSIONAL TOTAL STRESS ACCELERATION RESPONSE SPECTRA BORING BH-1-10

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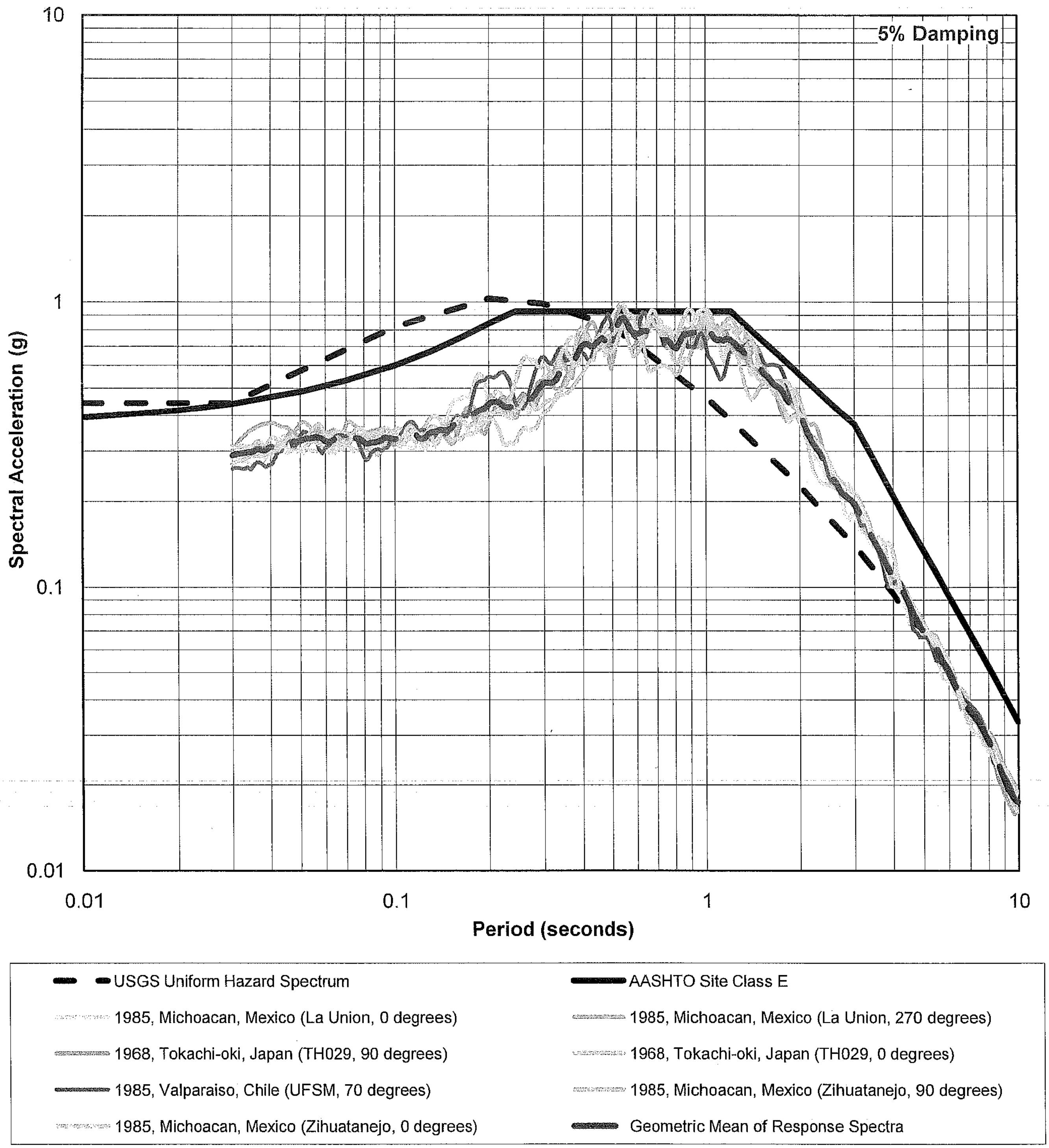
FIG. G-22



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ONE-DIMENSIONAL TOTAL STRESS ACCELERATION RESPONSE SPECTRA BORING BH-2-10

August 2010 21-1-21190-015

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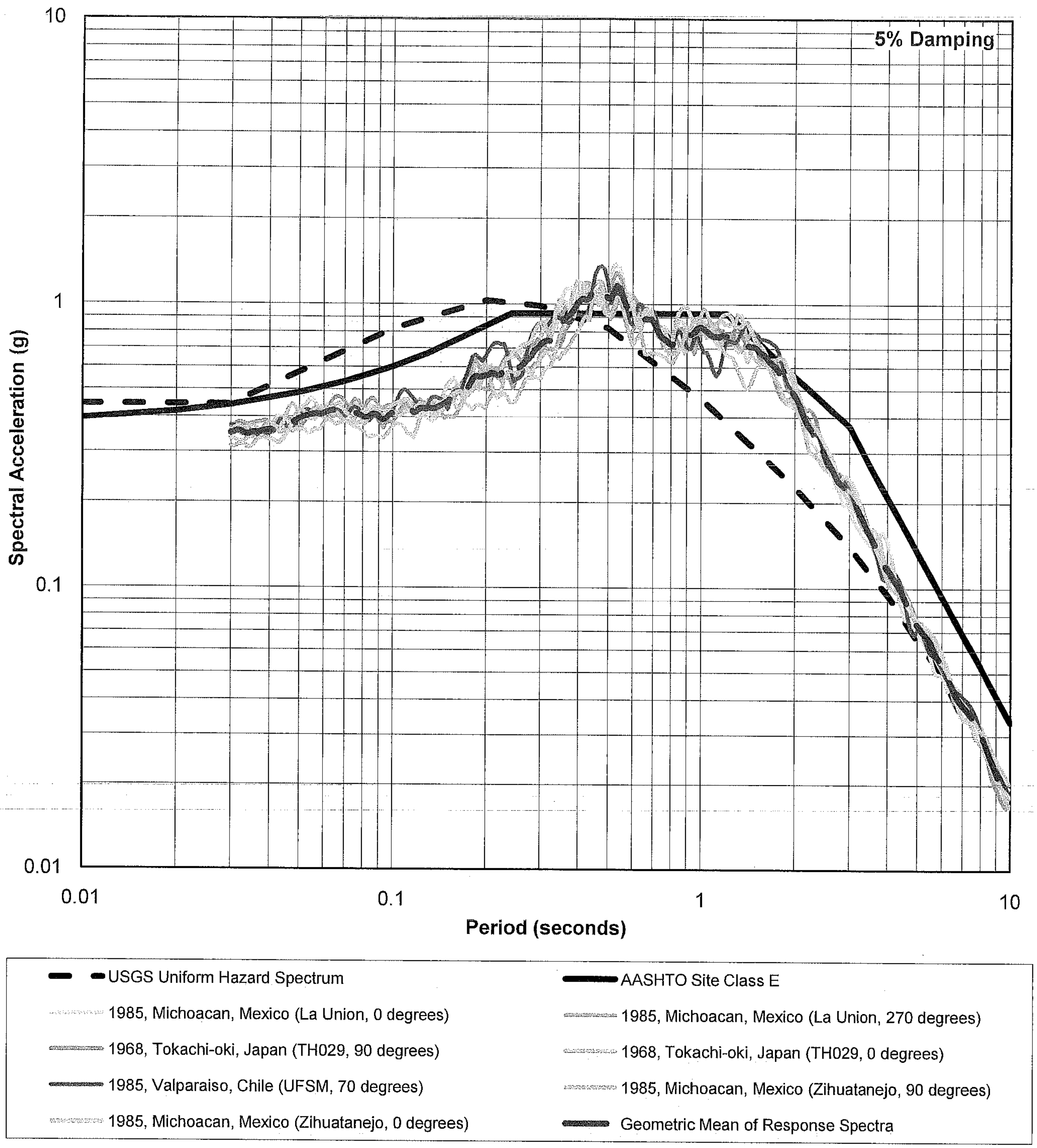
FIG. G-23



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ONE-DIMENSIONAL TOTAL STRESS ACCELERATION RESPONSE SPECTRA BORING H-07-09

August 2010 | 21-1-21190-015

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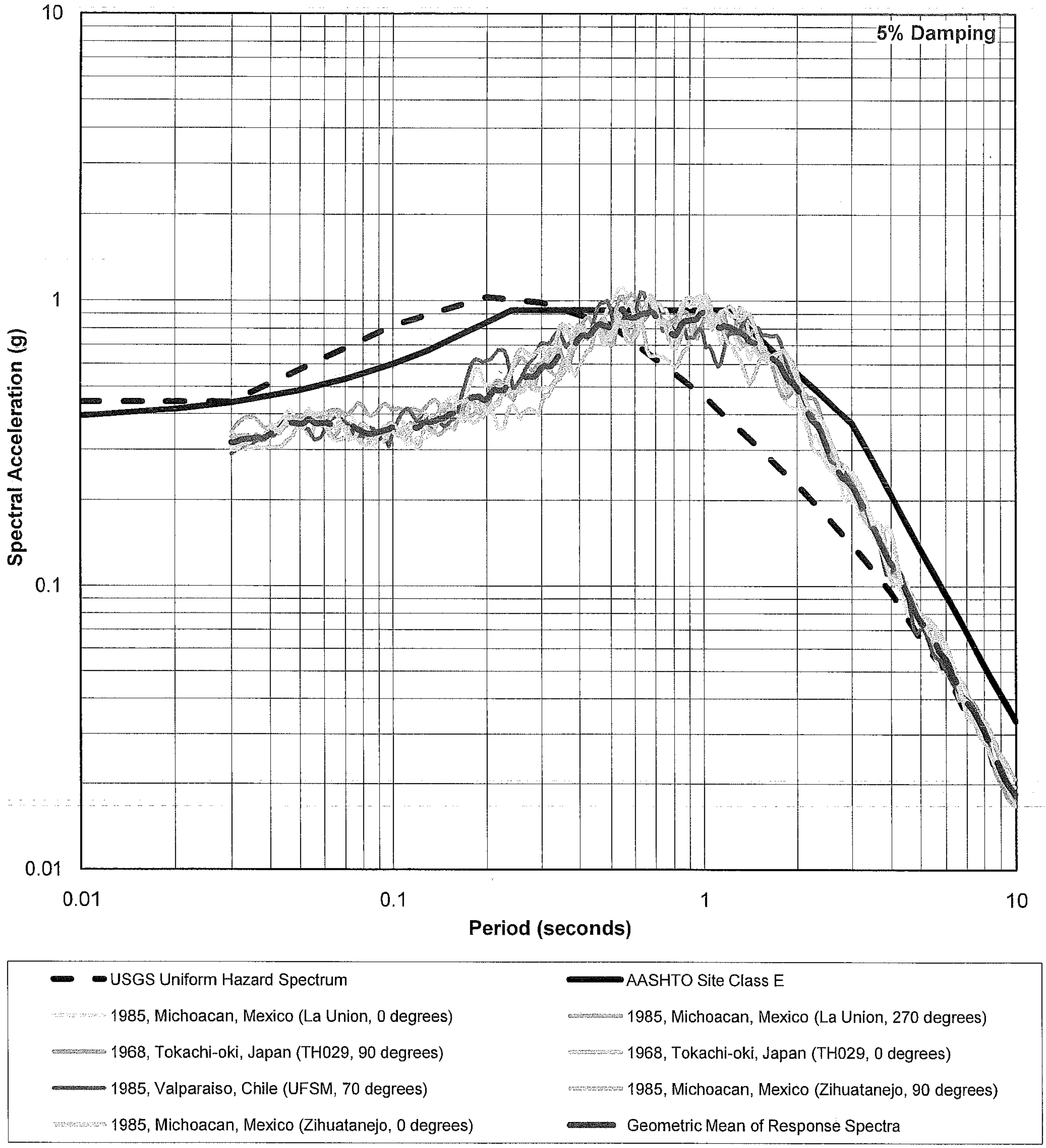
FIG. G-24

FLAC 1DSetupv3.23 (Tot Stress).xlsm 8/24/2010



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ONE-DIMENSIONAL TOTAL STRESS ACCELERATION RESPONSE SPECTRA BORING H-08-09

August 2010 | 21-1-21190-015

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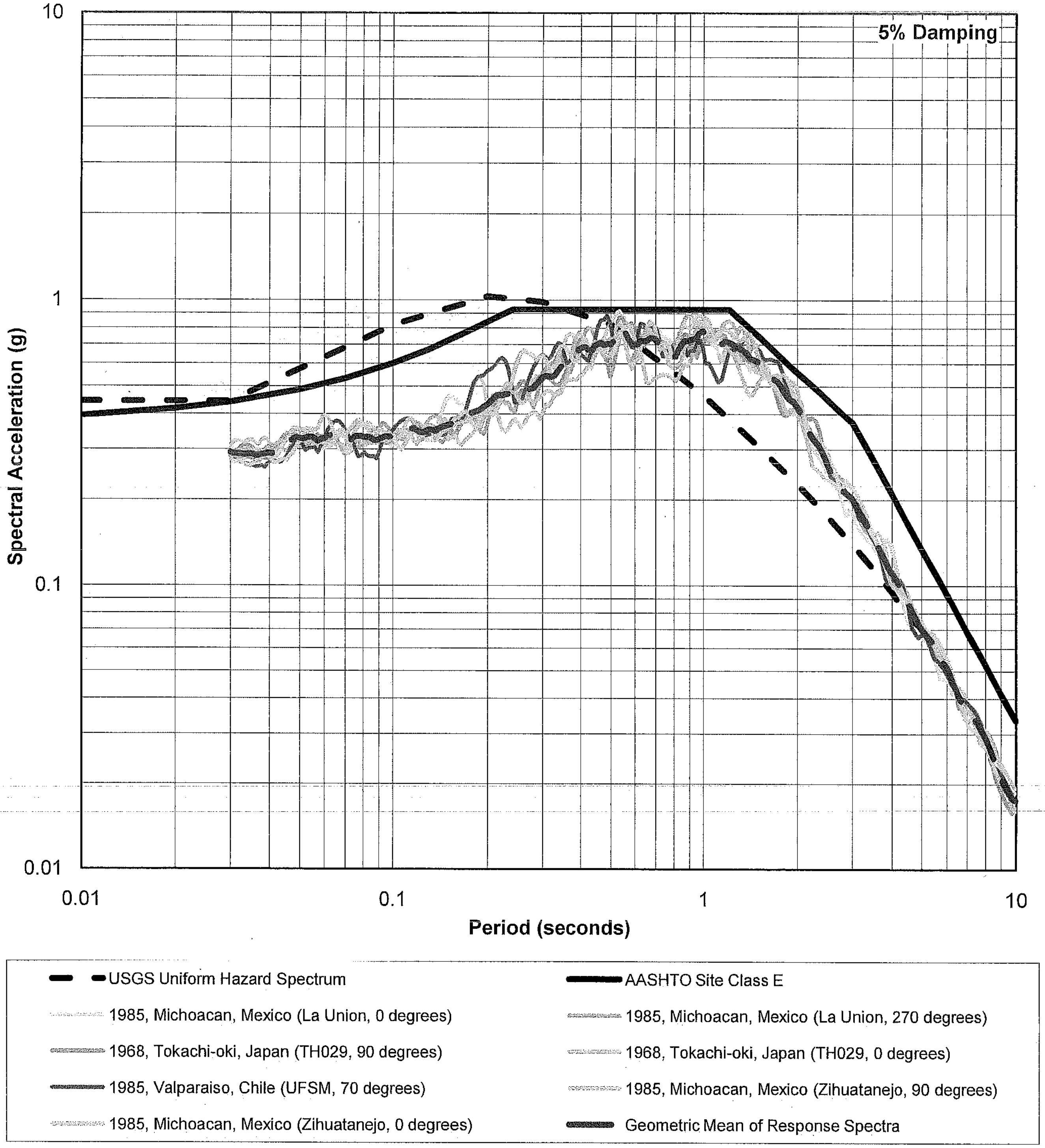
FIG. G-25

FLAC 1DSetupv3.23 (Tot Stress).xlsm 8/24/2010



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ONE-DIMENSIONAL TOTAL STRESS ACCELERATION RESPONSE SPECTRA BORING H-16-09

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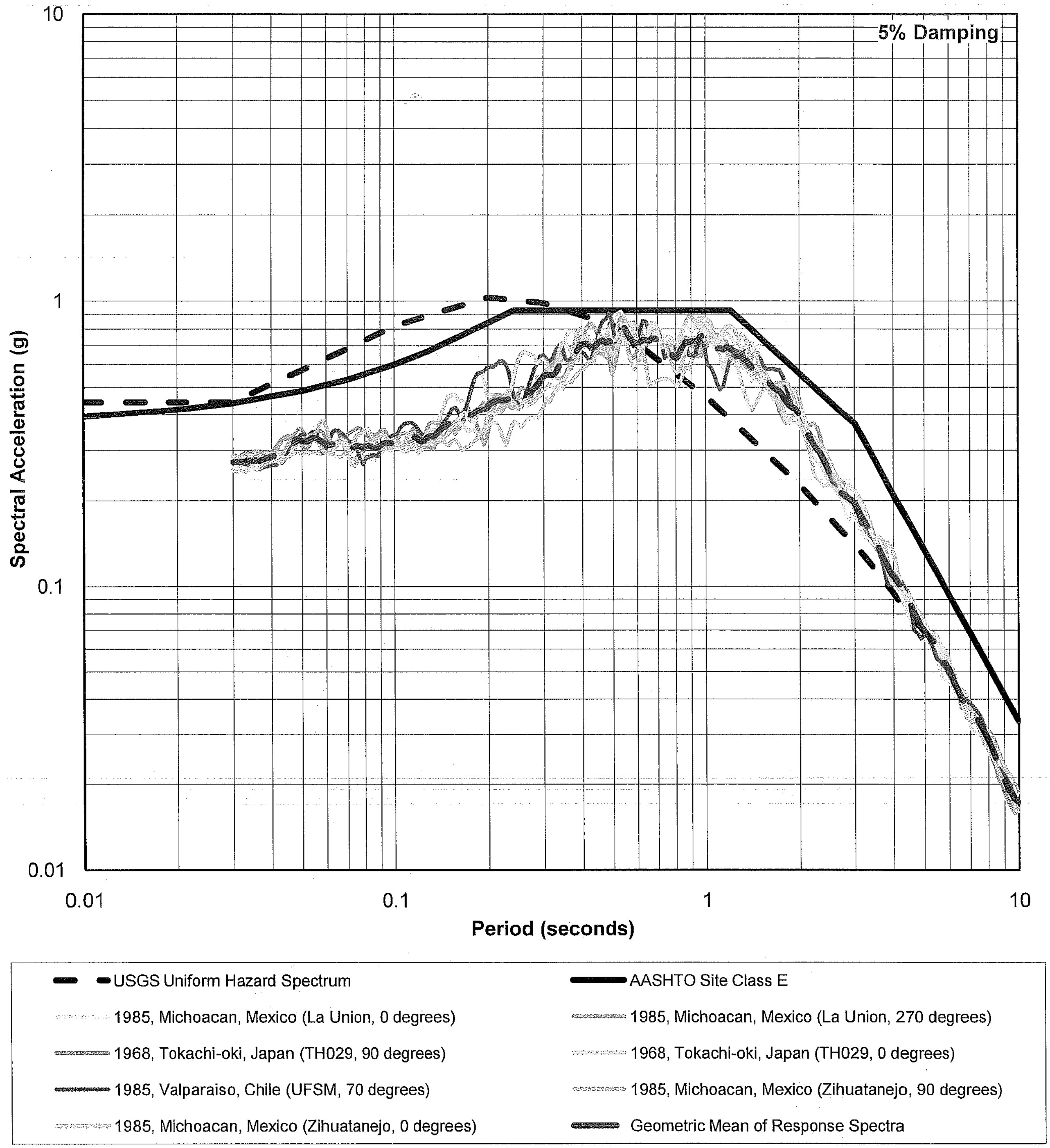
FIG. G-26



SR 520 Pontoon Casting Facility
Aberdeen, Washington

ONE-DIMENSIONAL TOTAL STRESS ACCELERATION RESPONSE SPECTRA BORING H-18P-09

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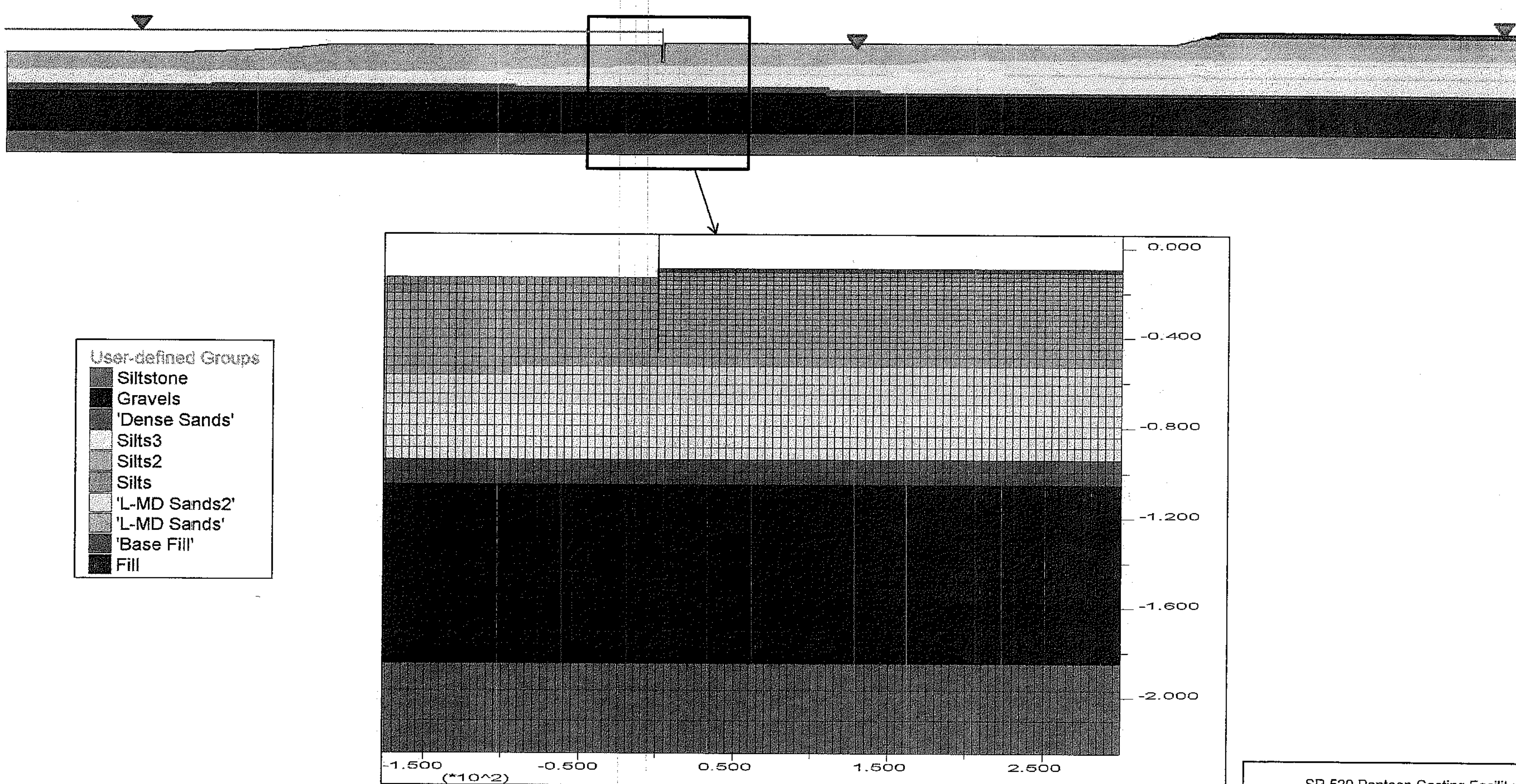
FIG. G-27

Model Geo - LC.xlsm Printed: 8/25/2010 12:25 PM



SR 520 Pontoon Casting Facility
Aberdeen, Washington

LONGITUDINAL (CENTER LINE)
MODEL GEOMETRY

August 2010 | 21-1-21190-016

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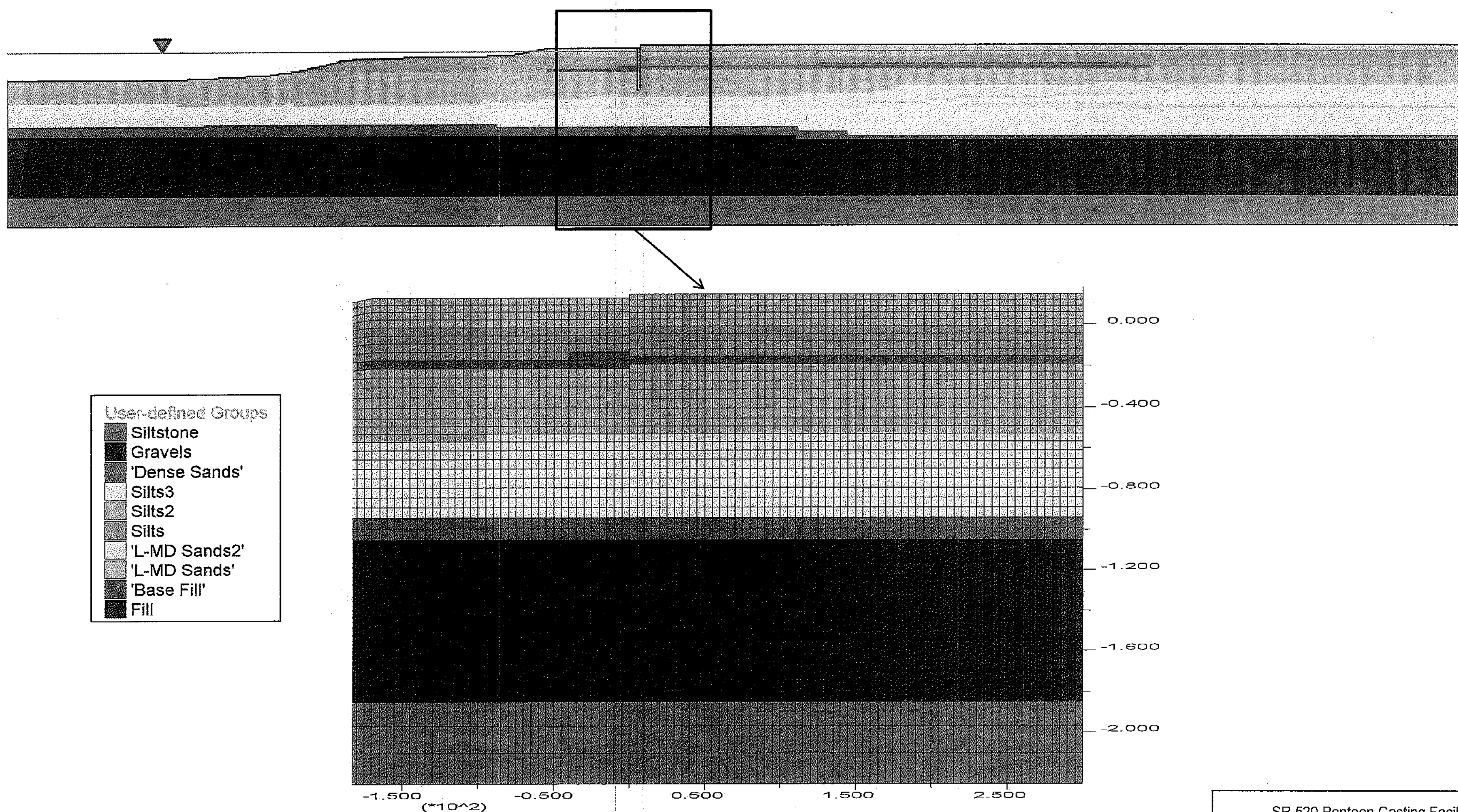
FIG. G-28

Model Geo - LS.xlsm Printed: 8/25/2010 12:24 PM



SR 520 Pontoon Casting Facility
Aberdeen, Washington

LONGITUDINAL (OUTSIDE BASIN)
MODEL GEOMETRY

August 2010 | 21-1-21190-016

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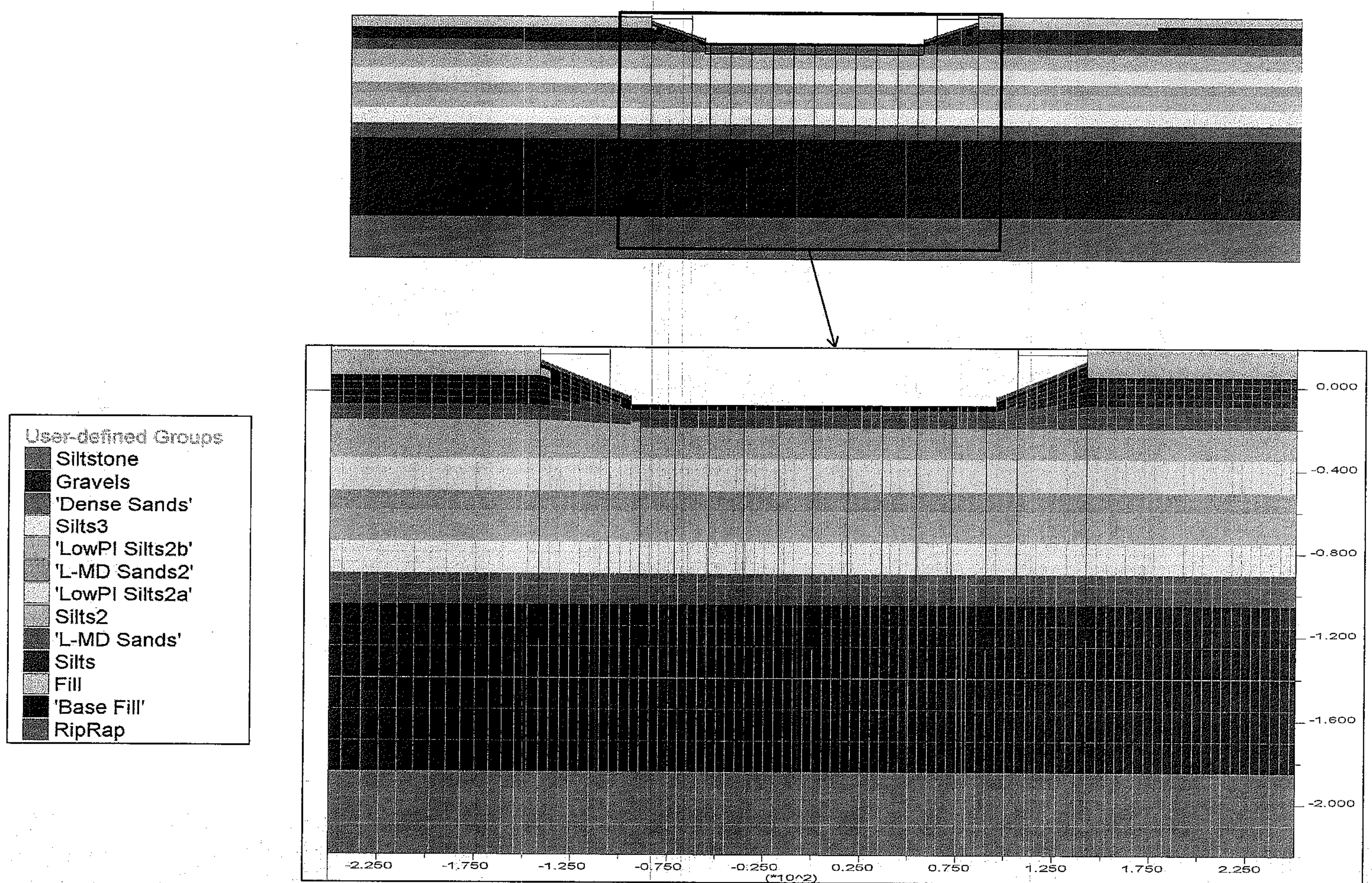
FIG. G-29



SR 520 Pontoon Casting Facility
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TRANSVERSE (NORTH BASIN)
MODEL GEOMETRY

December 2010 — 21-1-21190-016

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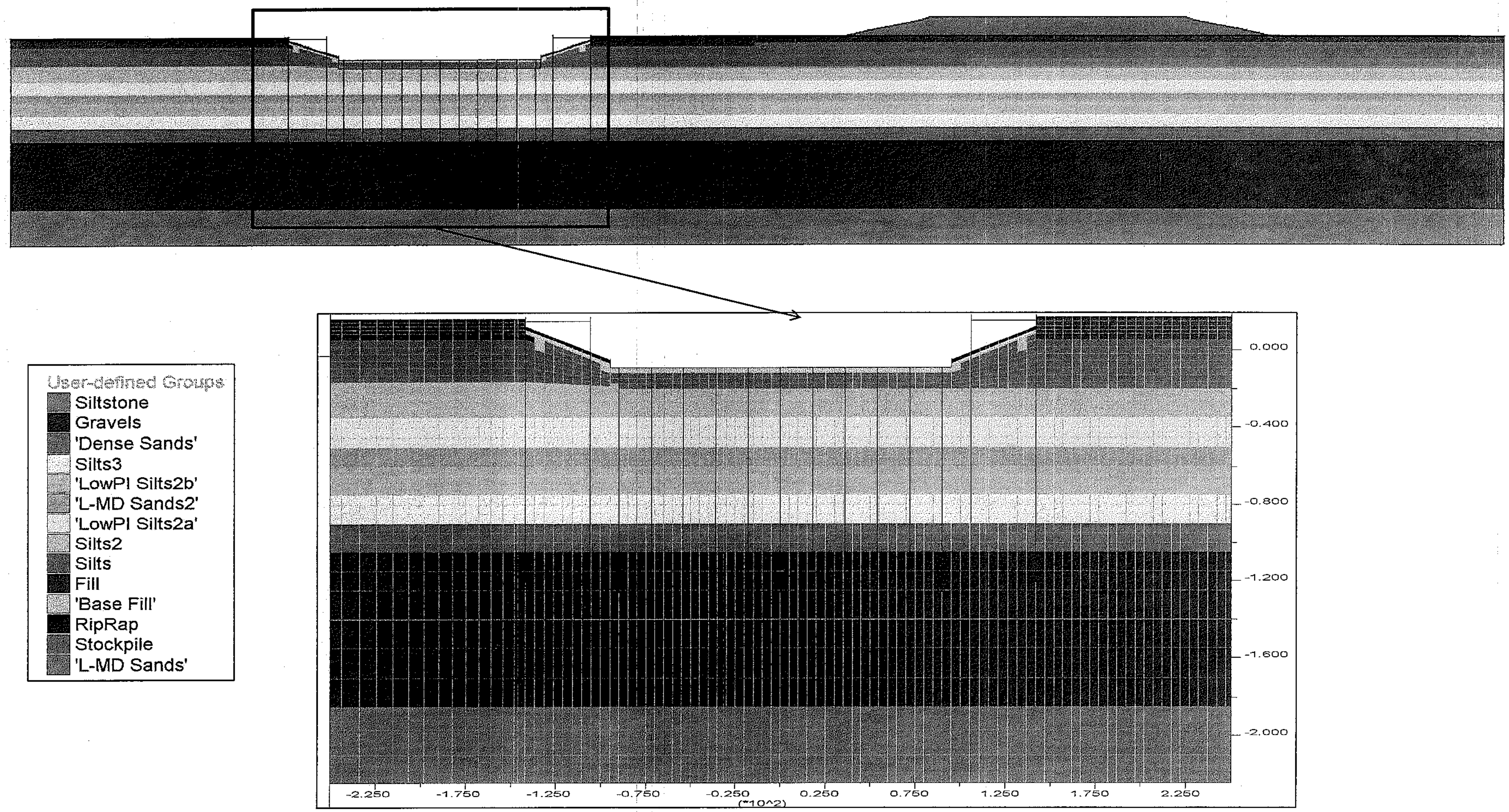
FIG. G-30

Model Geo -TsB.xlsm Printed: 8/25/2010 12:28 PM



SR 520 Pontoon Casting Facility
Aberdeen, Washington

TRANSVERSE (SOUTH BASIN)
MODEL GEOMETRY

August 2010 21-1-21190-016

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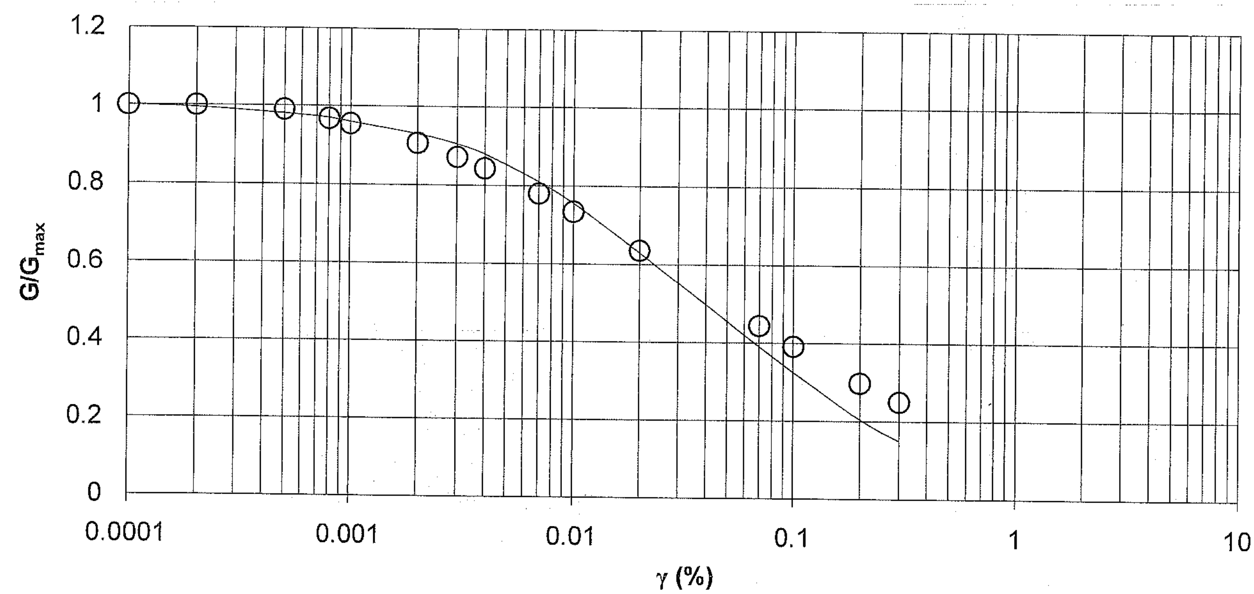
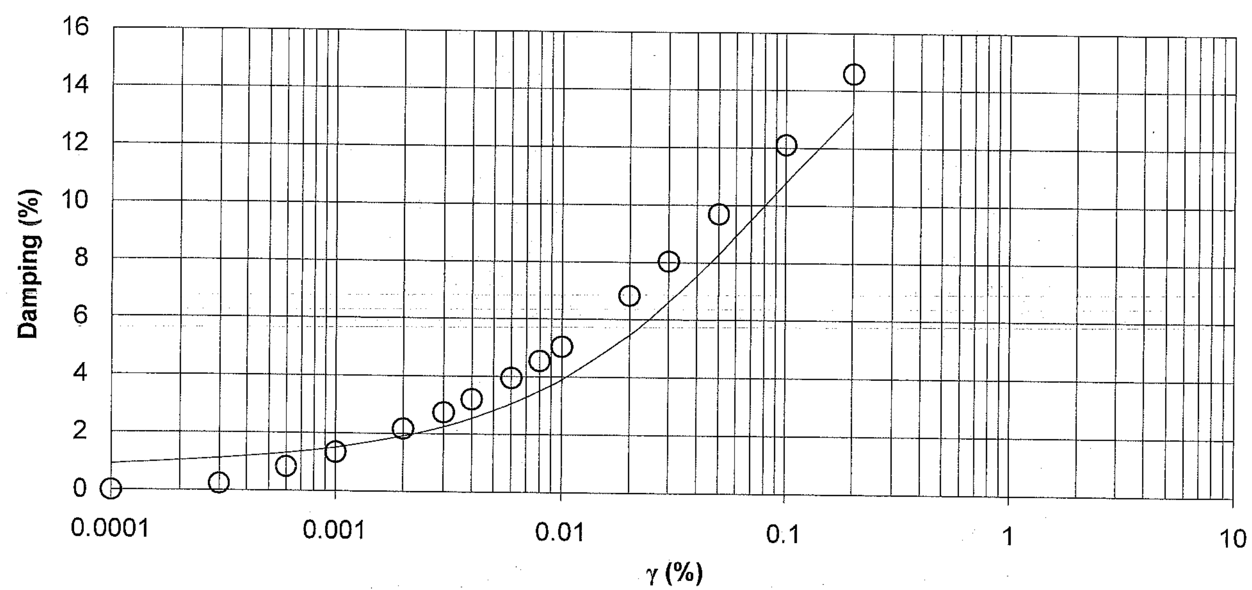
FIG. G-31





LEGEND: ———— TARGET CURVE   O BEST-FIT CURVE

NOTES

1. G/Gmax is the ratio of shear modulus to initial shear modulus; gamma is percent shear strain.
2. Best fit curves were determined by performing a series of virtual direct simple shear tests in a finite-difference computer program.
3. The computer program used was FLAC 6.0 (Fast Lagrangian Analysis of Continua), by Itasca Consulting Group (2009).
4. Four methods are available in FLAC to model hysteretic damping. The Sig3 model was used in this case. The best-fit parameters for the Sig3 model, as decribed in the FLAC manual (Section 3.4.2.8) are a = a =1.07000; b = -0.75000; x0 = -1.41288.

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Aberdeen, Washington

FLAC HYSTERETIC DAMPING MODEL CALIBRATION CURVES
GRAVEL - ROLLINS ET AL. (1998)

August 2010 | 21-1-21190-016

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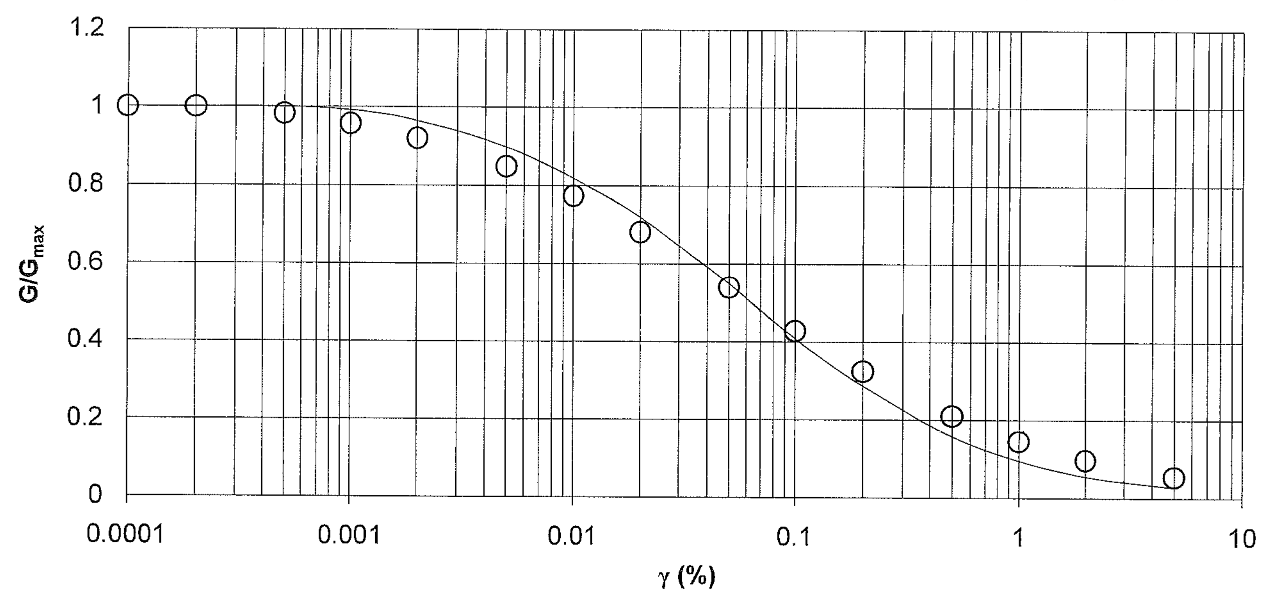
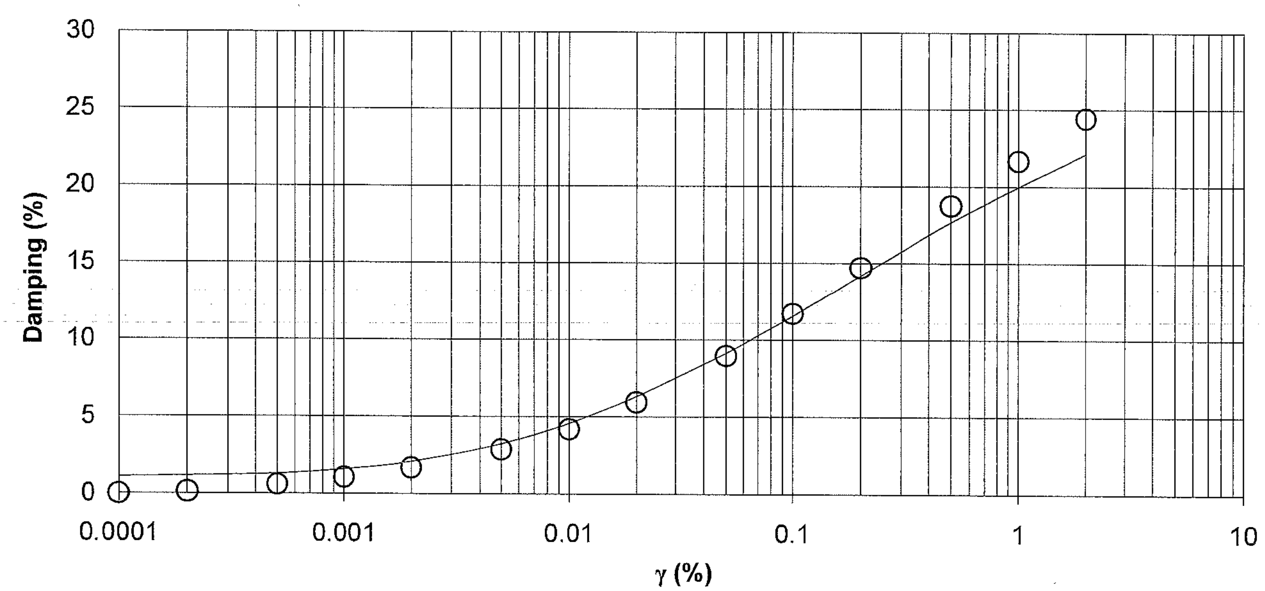
FIG. G-32

Printed: 8-25-10 12:28 File: I:\WP\21-1\21190 SR520 Casting Basin\Predesign\Analyses\Lateral Spread\Hysteretic\[CALIBRATE_HYSTv1.1 (Gravel Rollins).xls]Summary





LEGEND:	———— TARGET CURVE	O BEST-FIT CURVE

NOTES

1. G/Gmax is the ratio of shear modulus to initial shear modulus; gamma is percent shear strain.
2. Best fit curves were determined by performing a series of virtual direct simple shear tests in a finite-difference computer program.
3. The computer program used was FLAC 6.0 (Fast Lagrangian Analysis of Continua), by Itasca Consulting Group (2009).
4. Four methods are available in FLAC to model hysteretic damping. The Sig4 model was used in this case. The best-fit parameters for the Sig4 model, as decribed in the FLAC manual (Section 3.4.2.8) are a = a =1.04192; b = -0.70000; x0 = -1.23093; and y0 = -0.00728.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

FLAC HYSTERETIC DAMPING MODEL CALIBRATION CURVES VUCETIC & DOBRY (PI=15)

August 2010 21-1-21190-016

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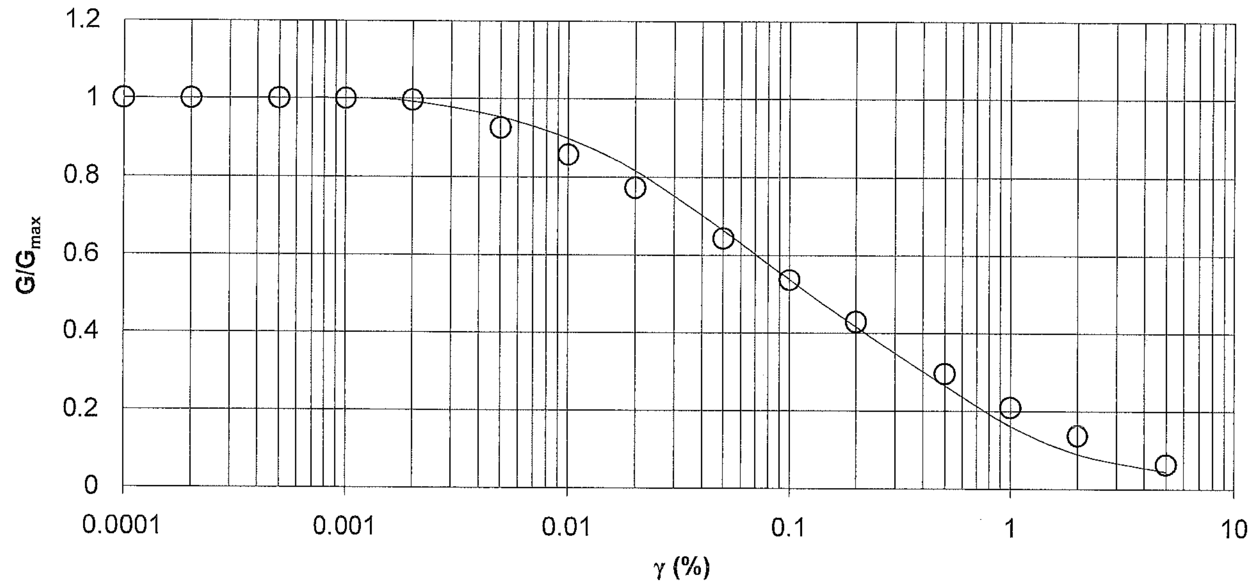
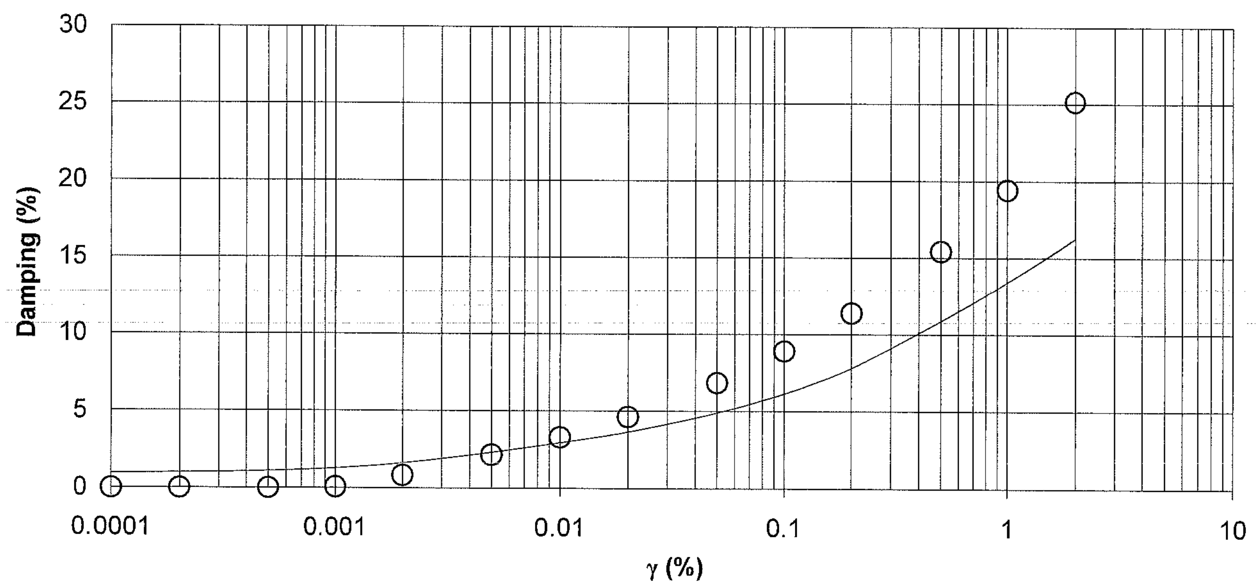
FIG. G-33





LEGEND: ———— TARGET CURVE   O BEST-FIT CURVE

NOTES

1. G/Gmax is the ratio of shear modulus to initial shear modulus; gamma is percent shear strain.
2. Best fit curves were determined by performing a series of virtual direct simple shear tests in a finite-difference computer program.
3. The computer program used was FLAC 6.0 (Fast Lagrangian Analysis of Continua), by Itasca Consulting Group (2009).
4. Four methods are available in FLAC to model hysteretic damping. The Sig4 model was used in this case. The best-fit parameters for the Sig4 model, as decribed in the FLAC manual (Section 3.4.2.8) are a = a =1.22513; b = -0.85000; x0 = -0.96724; and y0 = -0.08755.

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Aberdeen, Washington

FLAC HYSTERETIC DAMPING MODEL CALIBRATION CURVES VUCETIC & DOBRY (PI=30)

August 2010 21-1-21190-016

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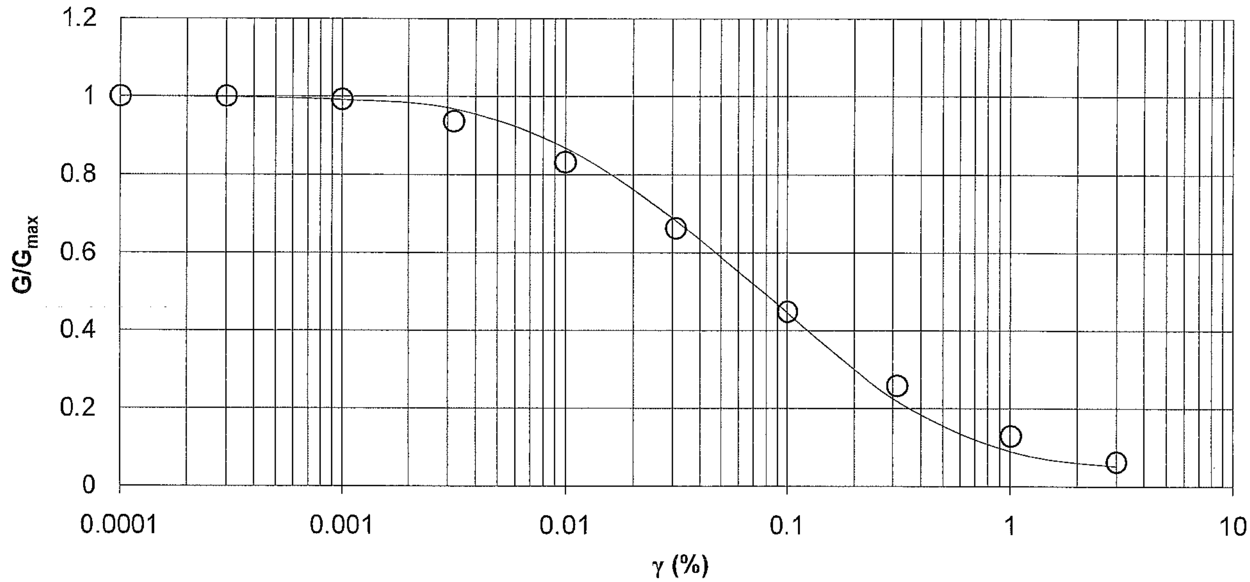
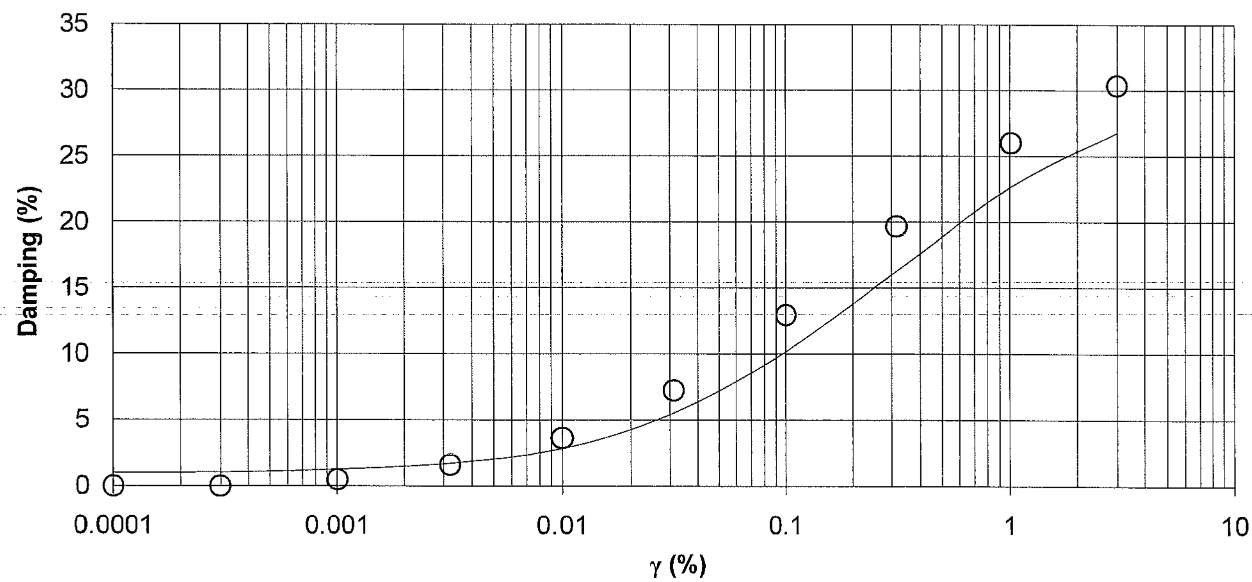
FIG. G-34





LEGEND: ———— TARGET CURVE O BEST-FIT CURVE

NOTES

1. G/Gmax is the ratio of shear modulus to initial shear modulus; gamma is percent shear strain.
2. Best fit curves were determined by performing a series of virtual direct simple shear tests in a finite-difference computer program.
3. The computer program used was FLAC 6.0 (Fast Lagrangian Analysis of Continua), by Itasca Consulting Group (2009).
4. Four methods are available in FLAC to model hysteretic damping. The Sig3 model was used in this case. The best-fit parameters for the Sig3 model, as decribed in the FLAC manual (Section 3.4.2.8) are a = a =1.03890; b = -0.60000; x0 = -1.16436.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

FLAC HYSTERETIC DAMPING MODEL CALIBRATION CURVES EPRI SOIL 15 to 36 METERS

August 2010 | 21-1-21190-016

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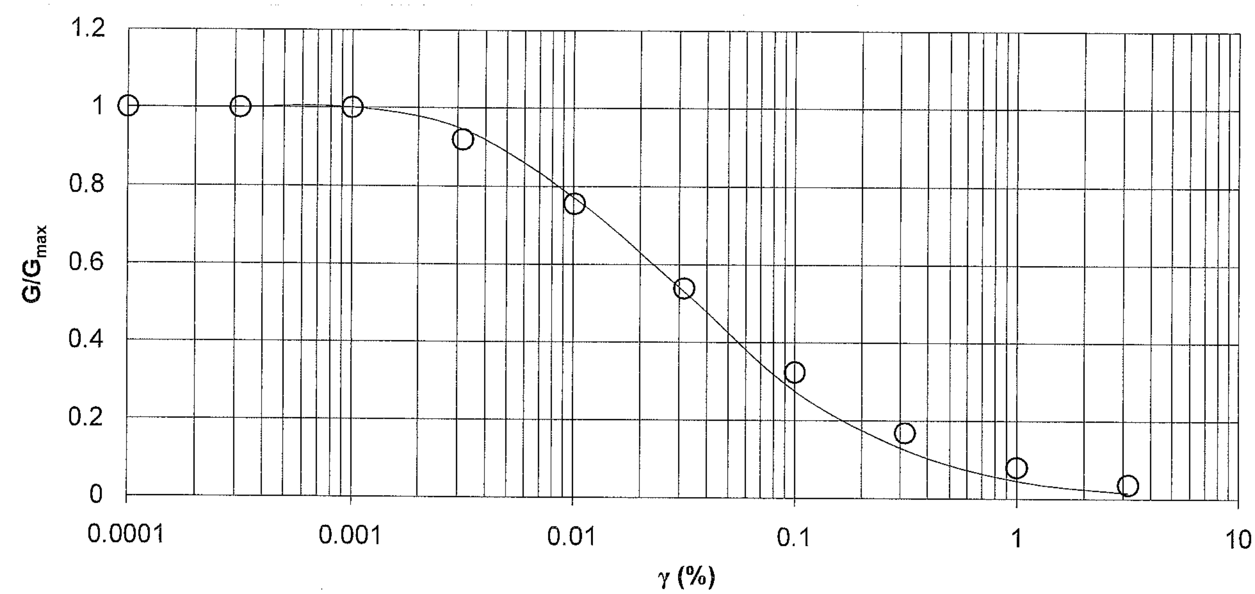
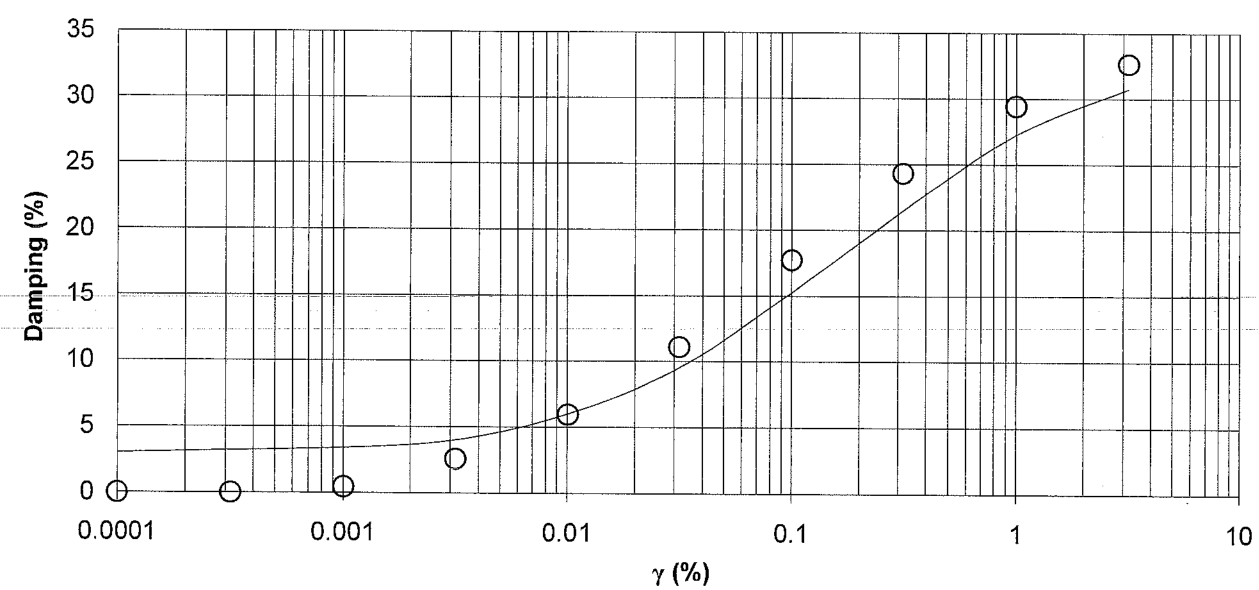
FIG. G-35





LEGEND: ———— TARGET CURVE O BEST-FIT CURVE

NOTES

1. G/Gmax is the ratio of shear modulus to initial shear modulus; gamma is percent shear strain.
2. Best fit curves were determined by performing a series of virtual direct simple shear tests in a finite-difference computer program.
3. The computer program used was FLAC 6.0 (Fast Lagrangian Analysis of Continua), by Itasca Consulting Group (2009).
4. Four methods are available in FLAC to model hysteretic damping. The Sig3 model was used in this case. The best-fit parameters for the Sig3 model, as decribed in the FLAC manual (Section 3.4.2.8) are a = a =1.10000; b = -0.60000; x0 = -1.52389.

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FLAC HYSTERETIC DAMPING MODEL CALIBRATION CURVES
ROCK - 251 TO 500 FEET (EPRI, 1993)

August 2010 21-1-21190-016

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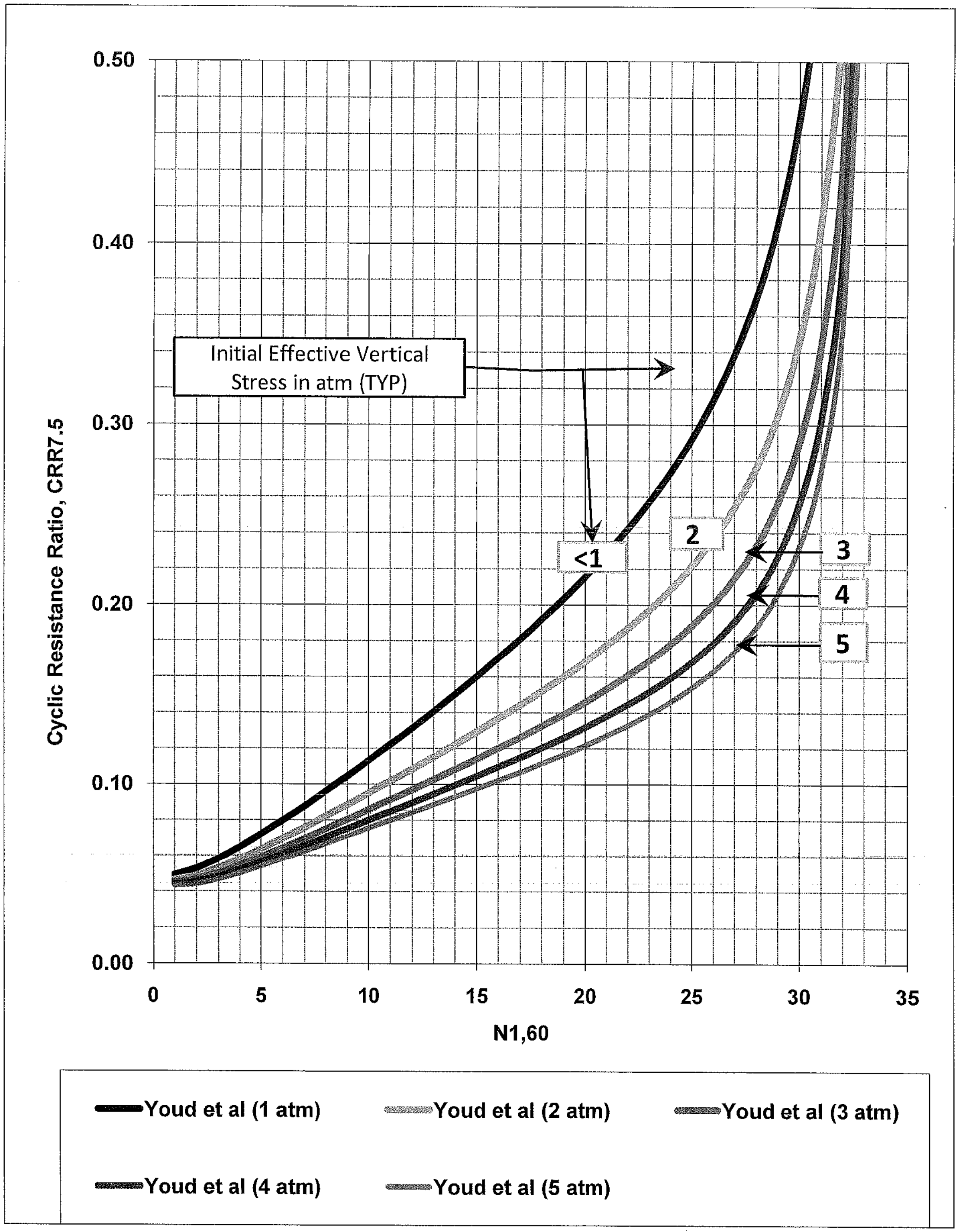
FIG. G-36



NOTES

1. UBCSAND Calibration of liquefaction triggering criteria based on Youd et al. (2001).

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UBCSAND CALIBRATION LIQUEFACTION TRIGGERING CRR VS N1,60

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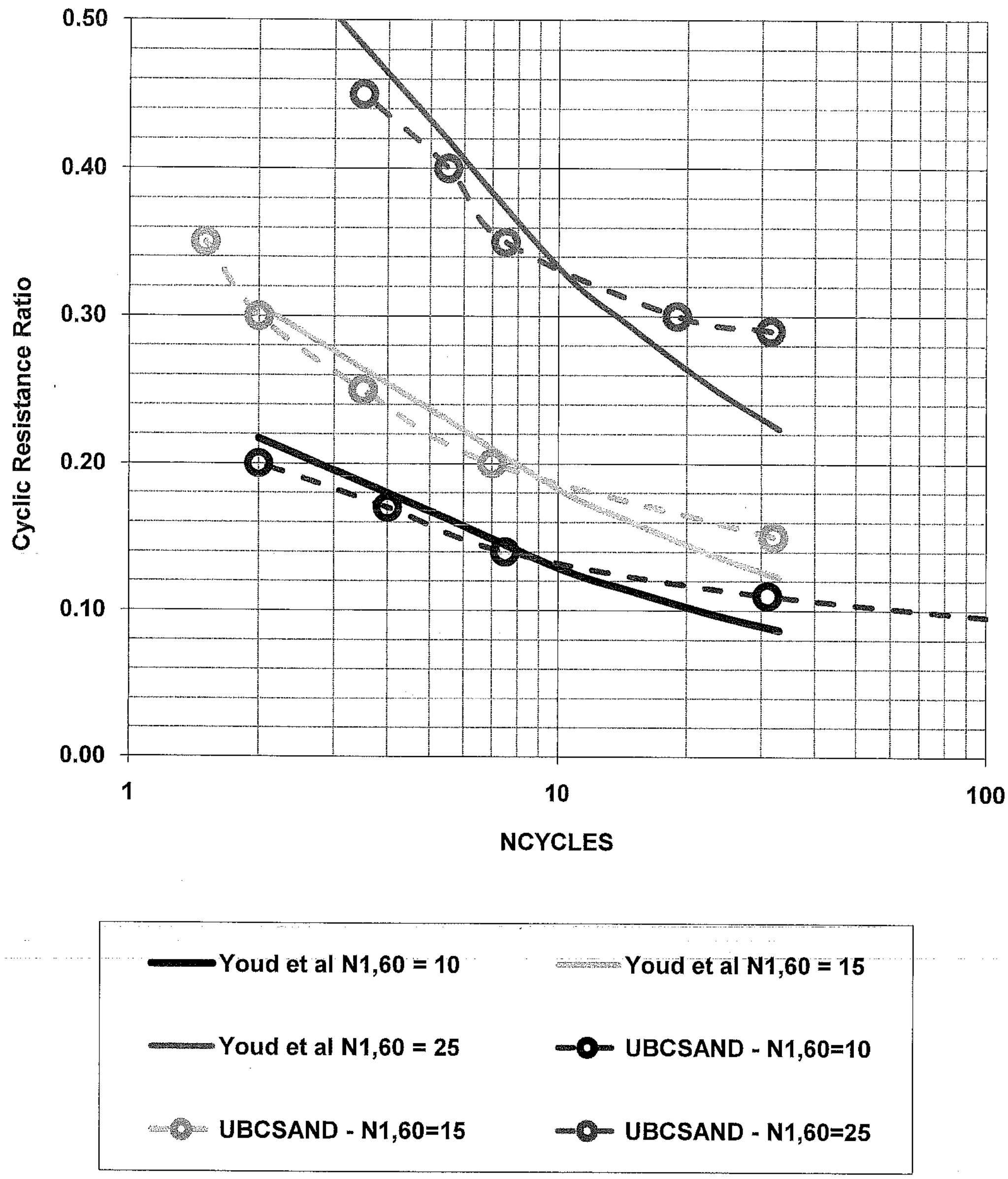
FIG. G-37

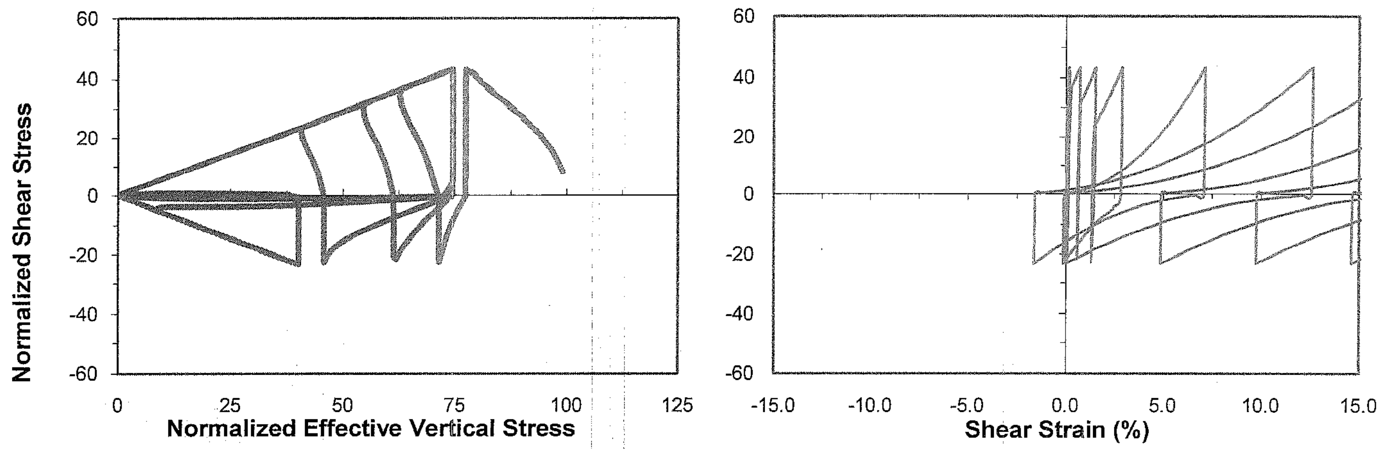


NOTES

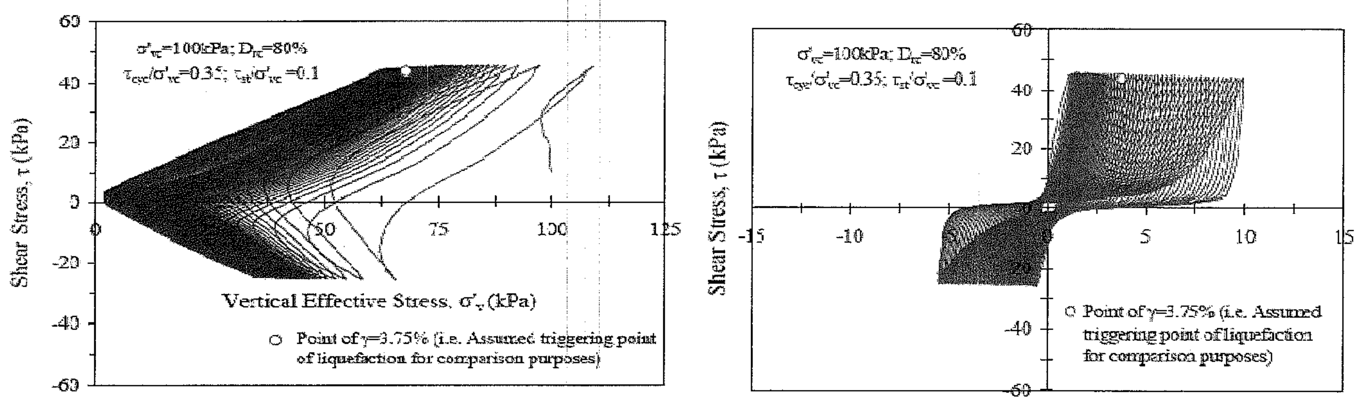
1. UBCSAND Calibration of liquefaction triggering criteria based on Youd et al. (2001).

SR 520 Pontoon Casting Facility Aberdeen, Washington	
UBCSAND CALIBRATION LIQUEFACTION TRIGGERING CRR VS NCYCLES	
August 2010	21-1-21190-016
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. G-38**

Printed: 8-25-10 12:35 File: I:\WIP\21-1\21190 SR520 Casting Basin\Predesign\Analyses\Lateral Spread\UBCSAND CAL\[CALIBRATE PLASTICITY_UBC15av3.1i.xlsm]Magnitude Figure G-11



UBCSAND



Fraser River Sand CDSS Laboratory Tests

NOTES

1. UBCSAND15A constitutive model was used in these simulations.
2. UBCSAND simulations were performed with the same input parameters as used in the CDSS Laboratory tests.

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UBCSAND POST-LIQUEFACTION LARGE STRAIN BEHAVIOR

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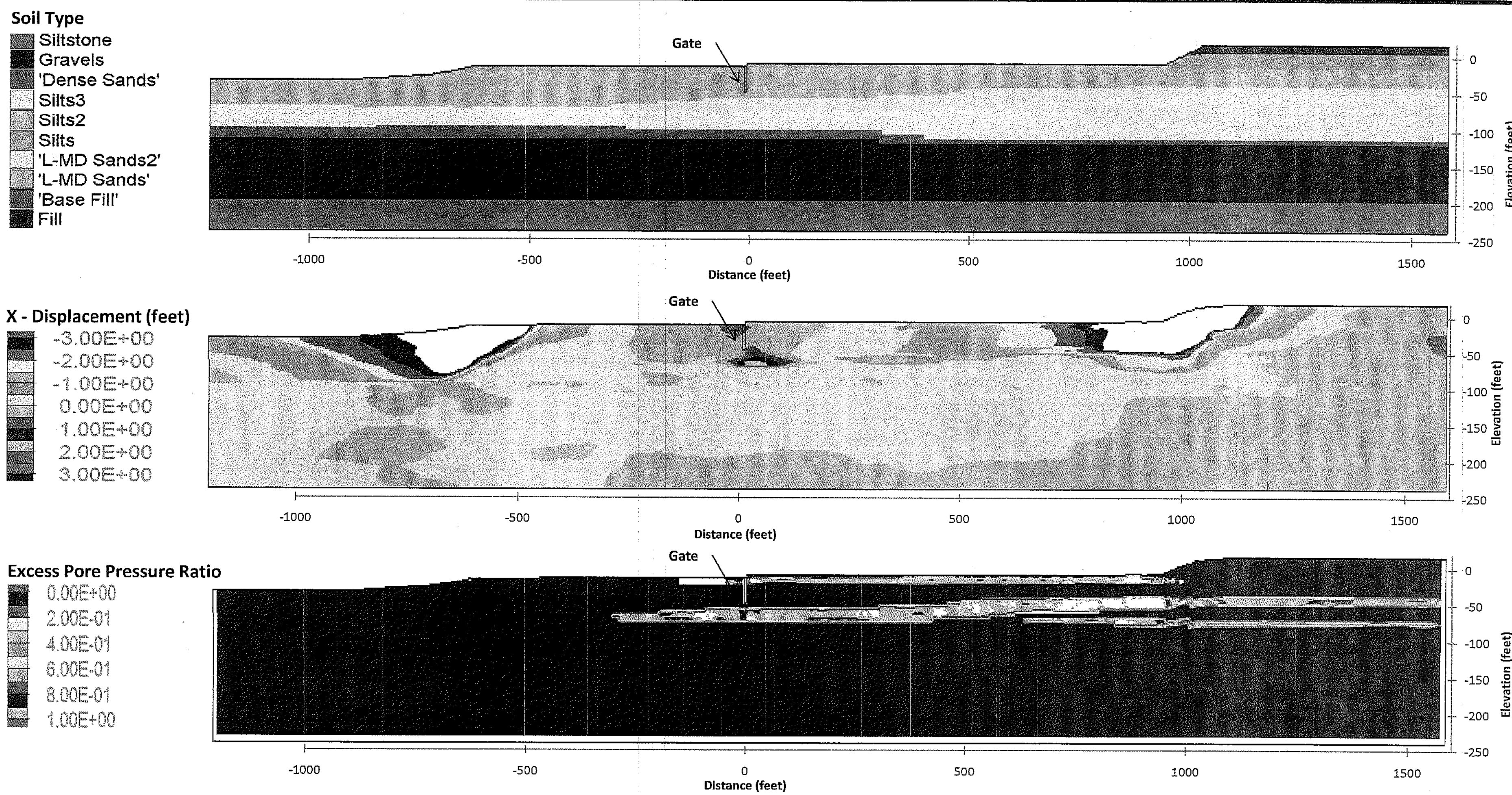
FIG. G-39

FIG. G-39

Figure G-40 through G-44.xlsm Printed: 8/29/2010 8:04 PM



NOTES

1. White contours indicate values greater than range plotted.

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LONGITUDINAL CENTERLINE OF BASIN HORIZONTAL DISPLACEMENT AND EXCESS PORE PRESSURE CONTOUR

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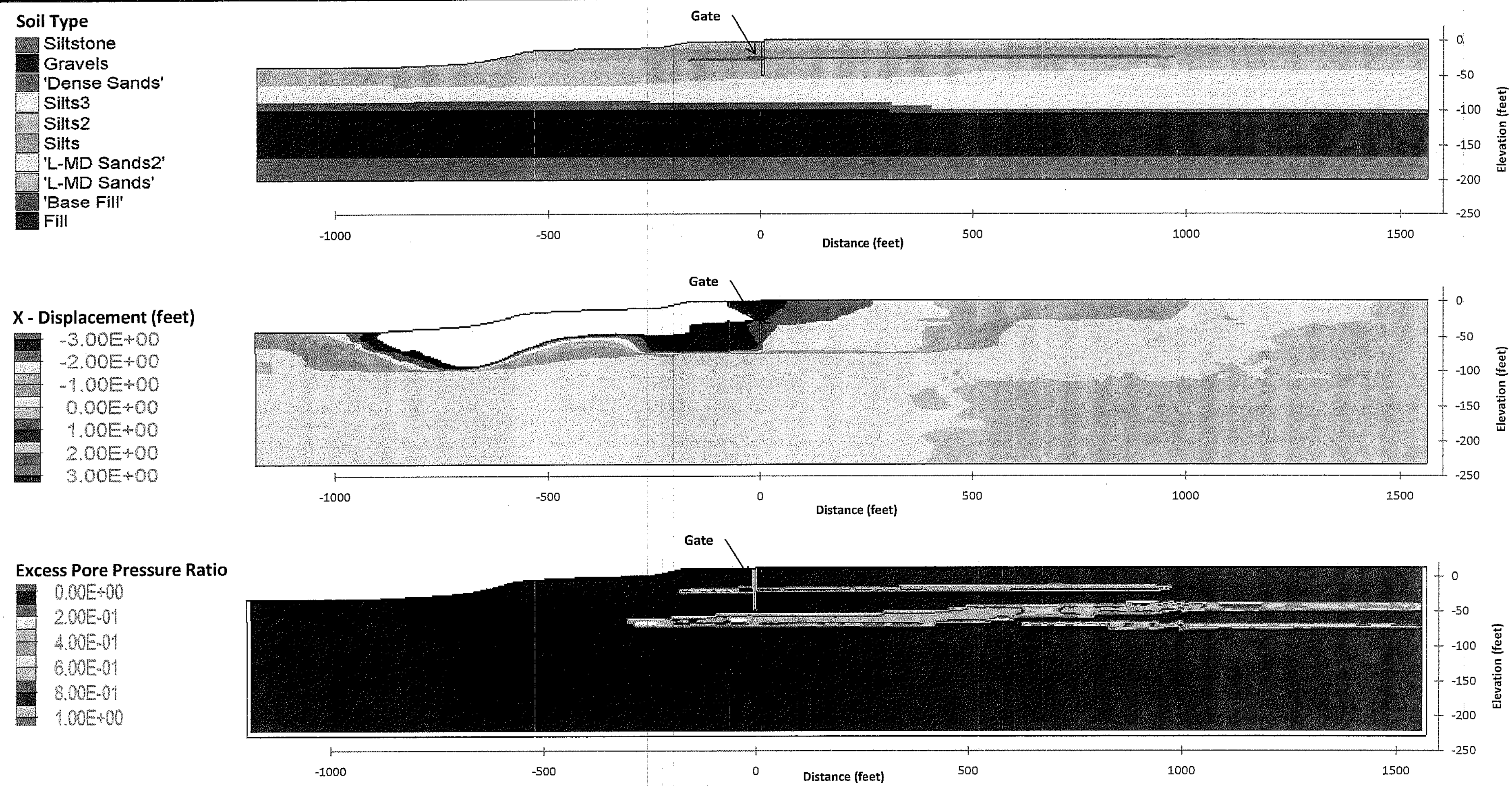
FIG. G-40

Figure G-40 through G-44.xlsm Printed: 8/29/2010 8:04 PM



NOTES

1. White contours indicate values greater than range plotted.

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LONGITUDINAL OUTSIDE OF BASIN HORIZONTAL DISPLACEMENT AND EXCESS PORE PRESSURE CONTOUR

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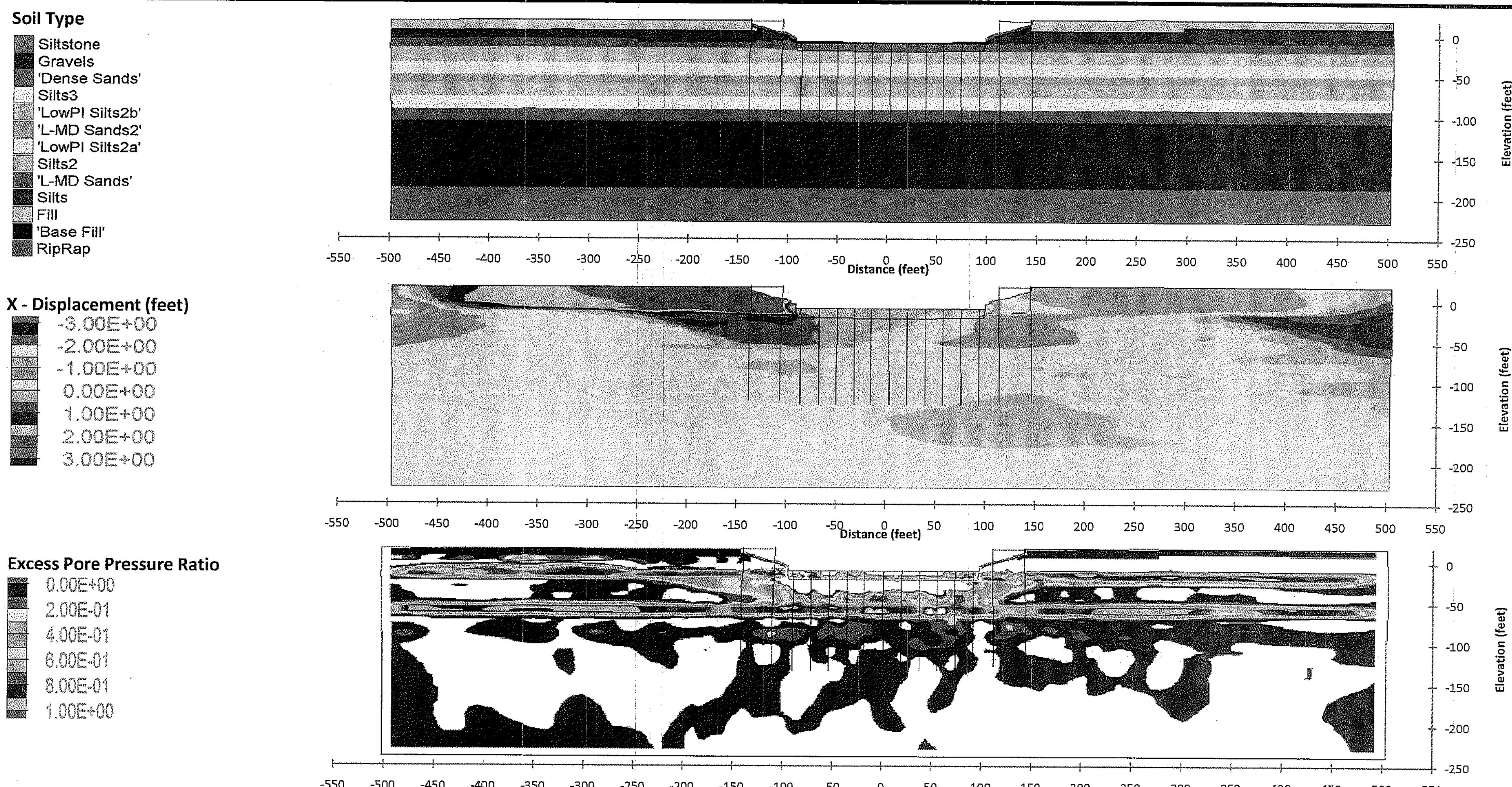
FIG. G-41



NOTES

1. White contours indicate values outside the range plotted. For horizontal displacements this means greater than 3 feet in either direction. For pore pressure ratio, this means greater than 1 or less than 0.

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TRANSVERSE NORTHERN HORIZONTAL DISPLACEMENT AND EXCESS PORE PRESSURE CONTOUR

December 2010 | 21-1-21190-015

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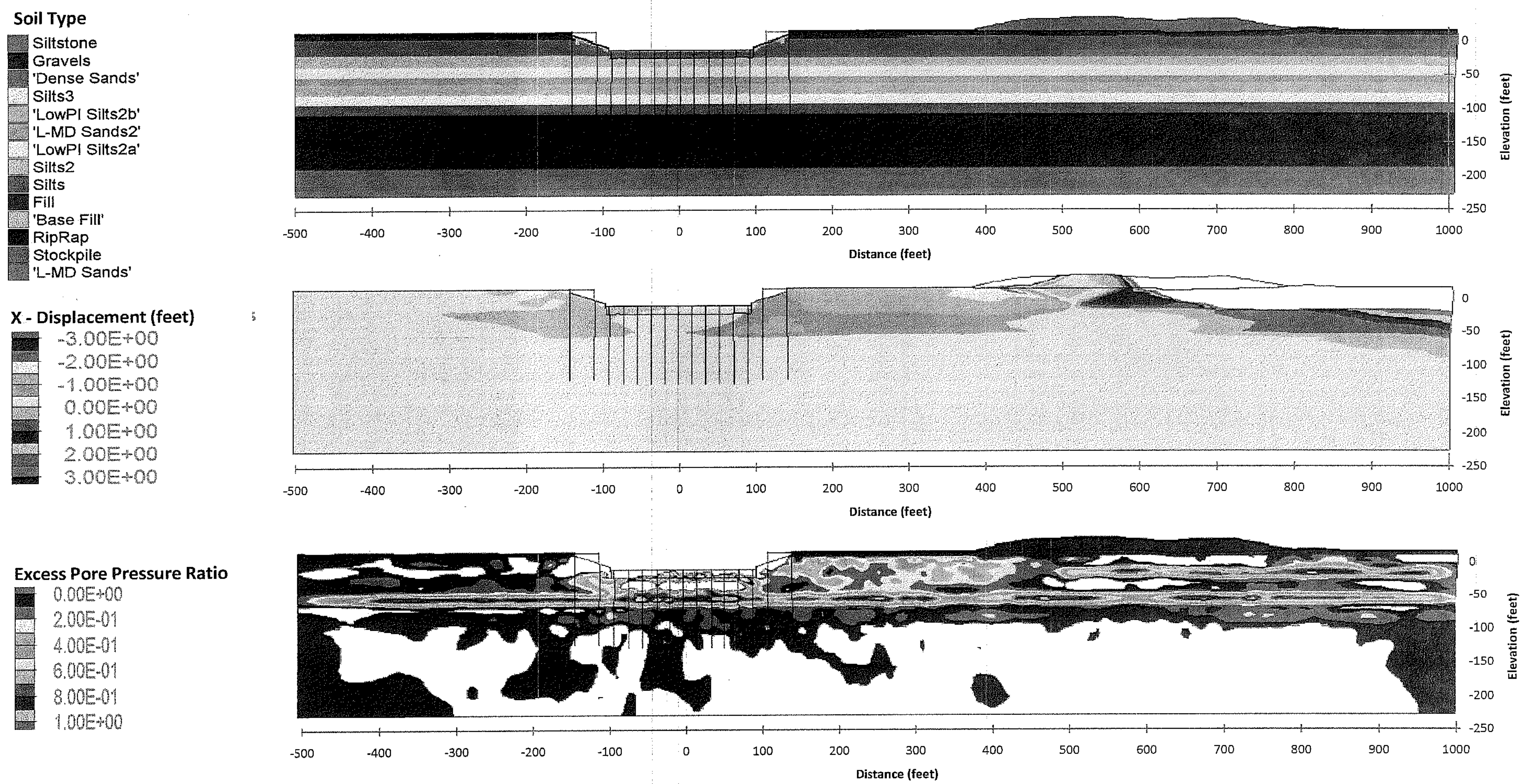
FIG. G-42

Figure G-40 through G-43.xlsm



NOTES

1. White contours indicate values outside the range plotted. For horizontal displacements this means greater than 3 feet in either direction. For pore pressure ratio, this means greater than 1 or less than 0.

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TRANSVERSE SOUTHERN HORIZONTAL DISPLACEMENT AND EXCESS PORE PRESSURE CONTOUR

December 2010 21-1-21190-015

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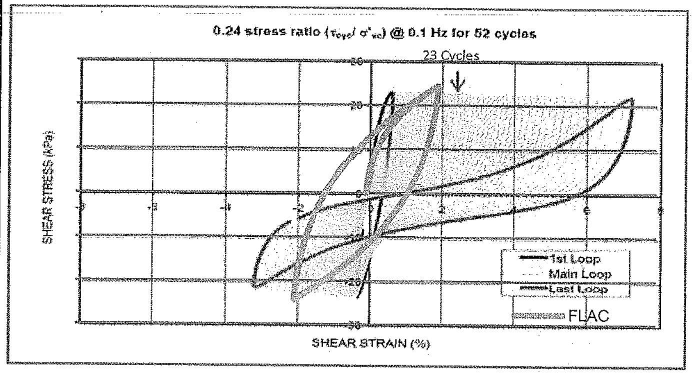
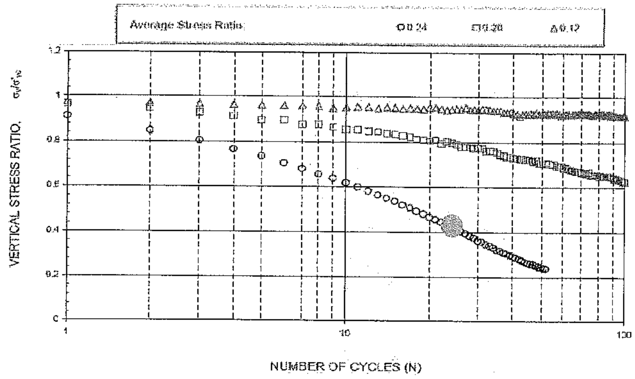
FIG. G-43

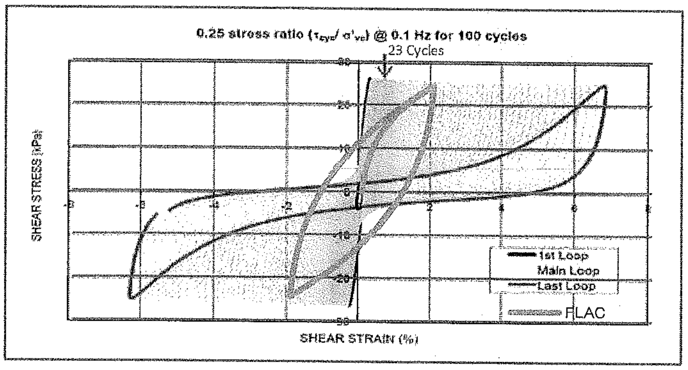
Boring H-8P-09

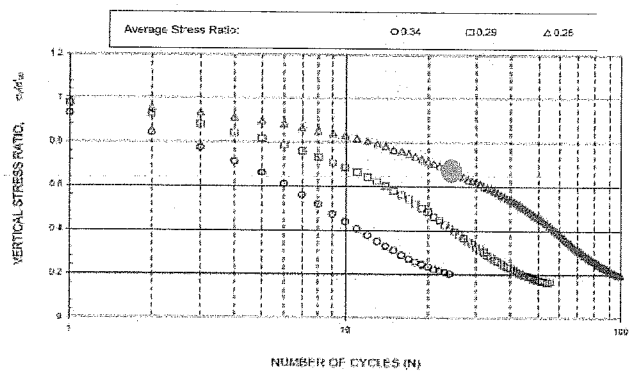




Ru = 0.68 @ CSR=0.24, Ncycles = 23 (Mw=8.3)

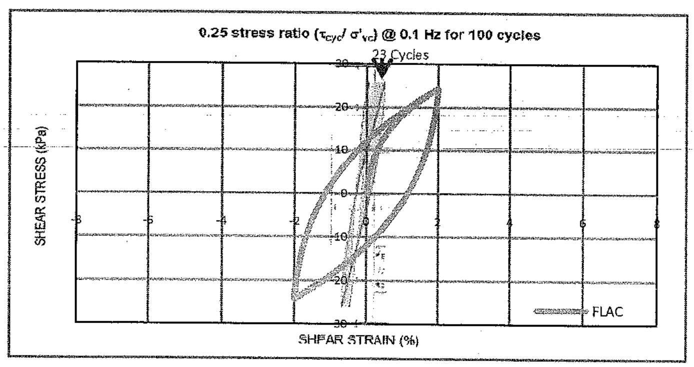
Boring H-18P-09

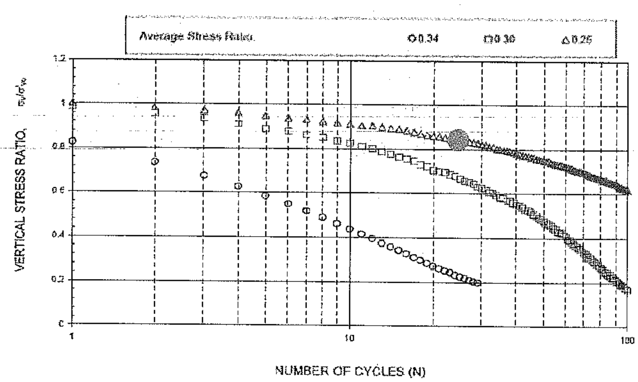




Ru = 0.35 @ CSR=0.25, Ncycles = 23 (Mw=8.3)

Boring H-7P-09





Ru = 0.16 @ CSR=0.25, Ncycles = 23 (Mw=8.3)

NOTES

1. CDSS test results provided by WSDOT in the reference documents.
2. Interpretation of CDSS results are centered on stress ratios of approximately 0.25, which were reflect the approximate results of the 2D site response.
3. FLAC hysteretic loops matched based CDSS tests shown above and Vucetic & Dobry (PI=15) modulus reduction curves.

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INTERPRETATION AND NUMERICAL APPROXIMATION OF CDSS TESTS FOR SILT (PI<17)

December 2010 21-1-21190-016

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FIG. G-44

APPENDIX H

HYDROGEOLOGIC TESTING AND ANALYSIS

APPENDIX H

HYDROGEOLOGIC TESTING AND ANALYSIS

TABLE OF CONTENTS

TABLES

FIGURES

FIGURES (cont.)

FIGURES (cont.)

APPENDIX H

HYDROGEOLOGIC TESTING AND ANALYSIS

H.1 PUMPING TESTS AND ANALYSIS

Shannon & Wilson performed deep and shallow pumping tests to evaluate the hydrogeologic conditions and dewatering feasibility at the site. We analyzed the pumping test data to estimate the following aquifer characteristics for use in our dewatering evaluation:

- **Hydraulic Conductivity** – The ability of a soil to transmit water. For the purposes of this report, hydraulic conductivity refers to the horizontal hydraulic conductivity.
- **Transmissivity** – The ability of an aquifer to transmit water and is equal to the aquifer hydraulic conductivity times the aquifer saturated thickness.
- **Storage Coefficient** – The volume of water released from a unit volume of saturated soil with a unit drop in hydraulic head.

We also performed infiltration testing to evaluate the infiltration capacity of shallow soils at the Pontoon Casting Facility (PCF) site. The following sections describe our pumping test program and results.

The results of previous pumping tests performed by the Washington State Department of Transportation were included in the Geotechnical Data Report. Those results were reviewed and considered in our analysis. The results of those tests are not included in this document.

H.1.1 Pumping and Monitoring Well Installation

Shannon & Wilson observed Slead Construction, under subcontract to Kiewit-General, drill and install two pumping wells and six monitoring wells at the PCF site between March 29 and April 8, 2010 (locations of pumping tests are shown in Figure 2 in the main text of the report). Figure H-1 shows the configuration of the two pumping wells (PW-3-10 and PW-4-10) and six monitoring wells (MW-1-10 through MW-6-10). Slead drilled the boreholes for the pumping wells with a 36-inch-diameter bucket auger rig, and the boreholes for the monitoring wells and vibrating wire piezometers (VWPs) using a 6-inch-diameter hollow-stem auger rig.

The monitoring wells consist of a 2-inch-diameter polyvinyl chloride (PVC) well casing with 10 feet of well screen. The pumping wells consist of a 12-inch-diameter PVC well casing with 20 feet of well screen. The well screens for the monitoring and pumping wells have 0.010-inch-wide slots (No. 10 slot) and are surrounded with a filter pack consisting of No. 10-20 silica sand. The VWPs (Geokon Model No. 4500S, 350 kPa) were installed in a bentonite-cement grout.

The pumping well screen and monitoring well screen/VWP depths are as follows:

- Pumping well PW-3-10: screened 15 to 35 feet below ground surface (bgs)
- Pumping well PW-4-10: screened 45 to 65 feet bgs
- Monitoring wells MW-1-10, MW-4-10, and MW-5-10: screened 15 to 35 feet bgs, VWP located at 65 feet bgs
- Monitoring wells MW-2-10, MW-3-10, and MW-6-10: screened 45 to 65 feet bgs, VWP located at 35 feet bgs

Figures H-2 through H-4 show schematic diagrams with installation details for the pumping and monitoring wells.

Slead developed the pumping wells by pumping and surging water through the well screen and the monitoring wells by using a bailer.

H.1.2 Pumping Tests

We performed three pumping tests in April 2010, including two tests in pumping well PW-3-10 and one test in pumping well PW-4-10. Each pumping test consisted of a step-rate test to estimate the target pumping rate and a constant-rate test to estimate the parameters of the aquifer. The tests also included evaluating the recovery of water levels after pumping which provides additional data for estimating the parameters of the aquifer.

The tests included:

- PW-3-10 Test 1, 52-hour constant-rate pumping test at 3.5 gallons per minute (gpm).
- PW-4-10 Test, 24-hour constant-rate pumping test at 7 gpm.
- PW-3-10 Test 2, 40-hour constant-rate pumping test at 10 gpm.

Groundwater level data was collected electronically using pressure transducer/datalogger systems in the monitoring wells (Levelogger Gold), and dataloggers attached to the VWPs (Geokon GK-404, LC-2). Hand measurements were also collected to confirm the data collected with the dataloggers.

Groundwater produced from the pumping tests discharged to a 20,000-gallon, two-weir settlement tank before discharging to the infiltration test pit conveyed through a hose that was routed along the existing ground surface.

Tables H-1 through H-3 summarize the results of pumping tests PW-3-10 Test 1, PW-4-10, and PW-3-10 Test 2, respectively, including maximum drawdown in each monitoring well, and resulting aquifer parameters (transmissivity, hydraulic conductivity, and storage coefficient).

The following sections describe pumping test analysis methods and the resulting aquifer parameters for each pumping test.

H.1.3 Analysis Methods

We analyzed the pumping test data using the methods of Theis (1935) and Cooper and Jacob (1946), which include the following assumptions:

- The pumped aquifer is confined, homogeneous, isotropic, of uniform thickness, and of infinite areal extent.
- The pre-pumping water table surface is horizontal.
- The aquifer is pumped at a constant discharge rate.
- The pumping well penetrates the entire thickness of the aquifer.

In our opinion, though all of these assumptions are rarely met in practice, the Theis and Cooper-and-Jacob methods are appropriate for estimating aquifer parameters for this study. These analytical methods and their underlying assumptions and limitations are fully described in Theis (1935) and Cooper and Jacob (1946).

Figures H-5, H-6, and H-7 show the various pumping test plots for PW-3-10 Test 1, PW-4-10, and PW-3-10 Test 2, respectively.

The Cooper-Jacob Analysis method is performed by graphing the drawdown data on a semi-log scale. The drawdown data normally plots as a straight line and allows for the determination of delta s (change in drawdown over one log cycle) and t (time at zero drawdown). These values are then used to calculate hydraulic conductivity, transmissivity, and the storage coefficient.

The Theis graphical method involves matching a dimensionless, theoretical response, type-curve to the measured drawdown versus time (time-drawdown) data. Curve matching is performed by superimposing the measured time-drawdown data on the type curve and adjusting the overlay until most of the observed data points fall on the curve. A match point is selected and values of time and drawdown are substituted into the Theis equations to calculate transmissivity and storage coefficient.

The recovery analysis method uses a plot of residual drawdown (s', the positive change in head after pumping stops) versus the ratio of elapsed time since the start of pumping over elapsed time since the end of pumping (t/t') on a semi-log scale. Delta s (change in drawdown over one log cycle) is determined from this plot and used for calculating hydraulic conductivity, and transmissivity.

H.1.4 Results

The pumping test results indicate a poor hydraulic connection between the two zones being pumped by PW-3-10 and PW-4-10. The instrumentation at depths of 35 feet only responded to the shallow pumping tests PW-3-10 Tests 1 and 2. The deep instrumentation at 65 feet only responded to the deep pumping test in PW-4-10. See arithmetic plots H.5-1, H.5-2, H.6-1, H.6-2, H.7-1, and H.7-2 for a graphical representation of both shallow and deep instrumentation during the pumping tests.

Tables H-1 through H-3 summarize pumping test results with values of hydraulic conductivity, transmissivity, and the storage coefficient for each pumping test. Values of hydraulic conductivity are estimated by dividing transmissivity by the saturated thickness of the pumped aquifer, assumed to be 10 feet based on previous explorations and observed soil conditions during drilling for the 2010 pumping tests. The results of the PW-3-10 Test No. 1 analyses indicate that the hydraulic conductivity of the aquifer ranges from about 2.7×10^{-3} to 4.8×10^{-3} centimeters per second (cm/sec). The results of the PW-4-10 test analyses indicate that the hydraulic conductivity of the aquifer ranges from about 3.7×10^{-3} to 9.4×10^{-3} cm/sec. The results of PW-3-10 Test No. 2 analyses indicate the hydraulic conductivity of the aquifer ranges between 2.1×10^{-3} to 7.3×10^{-3} cm/sec.

H.2 INFILTRATION TESTING

We conducted infiltration tests on April 12 and 13, 2010, in the location shown in the Site and Exploration Plan (Figure 2). The infiltration test pit was about 12 feet long, 5 feet wide, and 15 feet deep, and backfilled with free-draining gravel. Slead drilled three 6-inch-diameter hollow-stem auger borings for monitoring well and VWP installation adjacent to three sides of the infiltration pit. Figure H-8 shows a layout of the infiltration pit and monitoring wells. Slead installed a 2-inch PVC monitoring well in IF-1-10, and one VWP each in IF-2-10 and IF-3-10. Figure H-9 shows a monitoring well schematic for IF-1-10 and Figure H-10 shows a VWP schematic for IF-2-10 and IF-3-10.

We introduced water to the infiltration test pit through a hose fed by gravity from the settlement tank at the pumping test area. The gravity-fed flow rates averaged about 10 gpm, and resulted in about 2.5 feet of water level rise in IF-1-10, IF-2-10, and IF-3-10. Figure H-11 is an arithmetic plot of head change versus time during the infiltration testing.

H.3 REFERENCES

Cooper, H.H., Jr., and Jacob, C.E., 1946, A Generalized Graphical Method for Evaluating Formation Constants and Summarizing Well Field History: Transactions, American Geophysical Union, vol. 27, no. 4.

Theis, C.V., 1935, The Relation Between the Lowering of the Piezometric Surface and the Rate and Duration of Discharge of a Well Using Ground Water Storage: Transactions, American Geophysical Union, Washington D.C., p. 518-524.

TABLE H-1
PUMPING TEST RESULTS, PW-3-10 TEST 1

Observation Point	Distance from Pumping Well, r (feet)	Maximum Drawdown (feet)	Theis Curve Matching Analysis			Cooper-Jacob Straight Line Method Analysis			Recovery Analysis	
			Transmissivity (square feet/day)	Hydraulic Conductivity K (feet/day)	Storage Coefficient	Transmissivity (square feet/day)	Hydraulic Conductivity K (feet/day)	Storage Coefficient, S	Transmissivity (square feet/day)	Hydraulic Conductivity K (feet/day)
MW-1-10 Shallow Well	35.5	2.2	134	13.4	0.004	137	13.7	0.003	137	13.7
MW-2-10 Shallow VWP	10.7	3.4	89	8.9	0.009	95	9.5	0.008	95	9.5
MW-3-10 Shallow VWP	26.8	2.6	107	10.7	0.007	88	8.8	0.007	154	15.4
MW-4-10 Shallow Well	51.0	2.2	89	8.9	0.005	112	11.2	0.004	103	10.3
MW-5-10 Shallow Well	11.3	3.6	54	5.4	0.016	77	7.7	0.014	62	6.2
MW-6-10 Shallow VWP	50.5	2.5	77	7.7	0.005	82	8.2	0.003	88	8.8

Notes:

1. MW = monitoring well; PW = pumping well; VWP = vibrating wire piezometer.
2. PW-3-10 pumping rate first test = 3.5 gallons per minute; Start time 12:20, April 9/Stop time 16:38, April 11; duration approximately 52 hours.
3. Drawdown measured at the end of constant-rate test. Barometric pressure and/or tidal fluctuations may have influenced drawdown.
4. Aquifer thickness, b = 10 feet.
5. Depth of instrumentation = 35 feet.

TABLE H-2
PUMPING TEST RESULTS, PW-4-10 TEST

Observation Point	Distance from Pumping well, r (feet)	Maximum Drawdown (feet)	Theis Curve Matching Analysis			Cooper-Jacob Straight Line Method Analysis			Recovery Analysis	
			Transmissivity (square feet/day)	Hydraulic Conductivity K (feet/day)	Storage Coefficient, S	Transmissivity (square feet/day)	Hydraulic Conductivity K (feet/day)	Storage Coefficient, S	Transmissivity (square feet/day)	Hydraulic Conductivity K (feet/day)
MW-1 Deep VWP	23.5	9.6	67	11.1	0.002	82	13.7	0.001	57	5.7
MW-2-10 Deep Well	12.7	11.7	39	6.5	0.003	63	10.5	0.002	44	4.4
MW-3-10 Deep Well	26.0	9.3	59	9.8	0.001	77	12.8	0.001	54	5.4
MW-4-10 Deep VWP	51.8	4.0	191	31.9	0.004	160	26.7	0.003	126	12.6
MW-5-10 Deep VWP	24.2	9.1	77	12.8	0.002	82	13.7	0.001	72	7.2
MW-6-10 Deep Well	50.5	6.3	90	15.0	0.001	107	17.8	0.001	93	9.3

Notes:

1. MW = monitoring well; PW = pumping well; VWP = vibrating wire piezometer.
2. PW-4 pumping rate = 10 gallons per minute; Start time 17:00, April 14/Stop time 16:44, April 15; duration approximately 24 hours.
3. Drawdown measured at the end of constant-rate test. Barometric pressure and/or tidal fluctuations may have influenced drawdown.
4. Aquifer thickness, b = 6 feet.
5. Depth of instrumentation = 65 feet.

TABLE H-3
PUMPING TEST RESEULTS, PW-3-10 TEST 2

Observation Point	Distance from Pumping well, r (feet)	Maximum Drawdown (feet)	Theis Curve Matching Analysis			Cooper-Jacob Straight Line Method Analysis			Recovery Analysis	
			Transmissivity (square feet/day)	Hydraulic Conductivity K (feet/day)	Storage Coefficient, S	Transmissivity (square feet/day)	Hydraulic Conductivity K (feet/day)	Storage Coefficient, S	Transmissivity (square feet/day)	Hydraulic Conductivity K (feet/day)
MW-1-10 Shallow Well	35.5	5.0	89	8.9	0.003	112	11.2	0.003	99	9.9
MW-2-10 Shallow VWP	10.7	7.7	67	6.7	0.017	206	20.6	0.054	53	5.3
MW-3-10 Shallow VWP	26.8	5.3	89	8.9	0.008	88	8.8	0.007	82	8.2
MW-4-10 Shallow Well	51.0	5.2	89	8.9	0.002	91	9.1	0.002	88	8.8
MW-5-10 Shallow Well	11.3	8.5	82	8.2	0.019	60	6.0	0.012	37	3.7
MW-6-10 Shallow VWP	50.5	5.2	60	6.0	0.005	63	6.3	0.004	75	7.5

Notes:

1. MW = monitoring well; PW = pumping well; VWP = vibrating wire piezometer.
2. PW-3-10 pumping rate second test = 7.0 gpm; Start time 07:55, April 27/Stop time 23:52, April 28; duration of approximately 40 hours.
3. Drawdown measured at the end of constant-rate test. Barometric pressure and/or tidal fluctuations may have influenced drawdown.
4. Aquifer thickness, b = 10 feet.
5. Depth of instrumentation = 35 feet.

TABLE H-4
SUMMARY OF GROUNDWATER MODEL LAYERS

Model Layer Number	Model Layer Elevation Range (Feet)			Horizontal Hydraulic Conductivity $K_{x,y}$ (feet/day)	Vertical Hydraulic Conductivity K_z (feet/day)	Anisotropy Ratio ($K_z/K_{x,y}$)	Soil Type	Description/Notes
1	10	to	15	0.3	0.08	0.3	Mixed fill	silty, sandy gravel with variable clay and wood
2	5	to	10	130	65	0.5	Wood fill	logs and/or saw dust with silt, sand, and gravel
3 through 7	-10	to	5	0.03	0.003	0.1	Silt	clayey silt with variable fine sand
8 through 12	-20	to	-10	6 to 12*	1.2 to 2.4	0.2	Sand	slightly silty to silty sand with silt interbeds; zone of pumping tests in pumping well PW-3-10
13 through 15	-40	to	-20	0.03	0.003	0.1	Silt	clayey silt with variable fine sand
16 through 18	-70	to	-40	7 to 15 in sand** 0.03 in silt	1.4 to 3 in sand 0.003 in silt	0.2 in sand 0.1 in silt	Sand interbedded with Silt	slightly silty to silty sand with silt interbeds; zone of pumping test in pumping well PW-4-10; sand interbedded with clayey silt
19	-100	to	-70	0.03	0.003	0.1	Silt	silt and clayey silt

Notes:

* Hydraulic conductivity range shown for sand aquifer from elevations -10 to -20 feet represent a subset of the range of values based on pumping test results in pumping well PW-3-10.

** Hydraulic conductivity value shown for intermittent sand aquifers from elevation -40 to -70 feet represent a subset of the range of values based on pumping test results in pumping well PW-4-10.

TABLE H-5
GROUNDWATER MODELING RESULTS, LOW CONDUCTIVITY

Flow Model Time Step	Time (days)	Number of Wells Pumping	Dewatering Well Discharge (gpm)	Cutoff Perimeter Trench Drain Discharge (gpm)	Total Discharge Wells and Trench Drain (gpm)
1	6	18	151	64	215
2	12	24	190	85	275
3	18	30	265	117	382
4	31	33	234	90	324
5	48		190	65	255
6	67		164	57	221
7	90		147	52	199
8	118		135	49	184

Note:

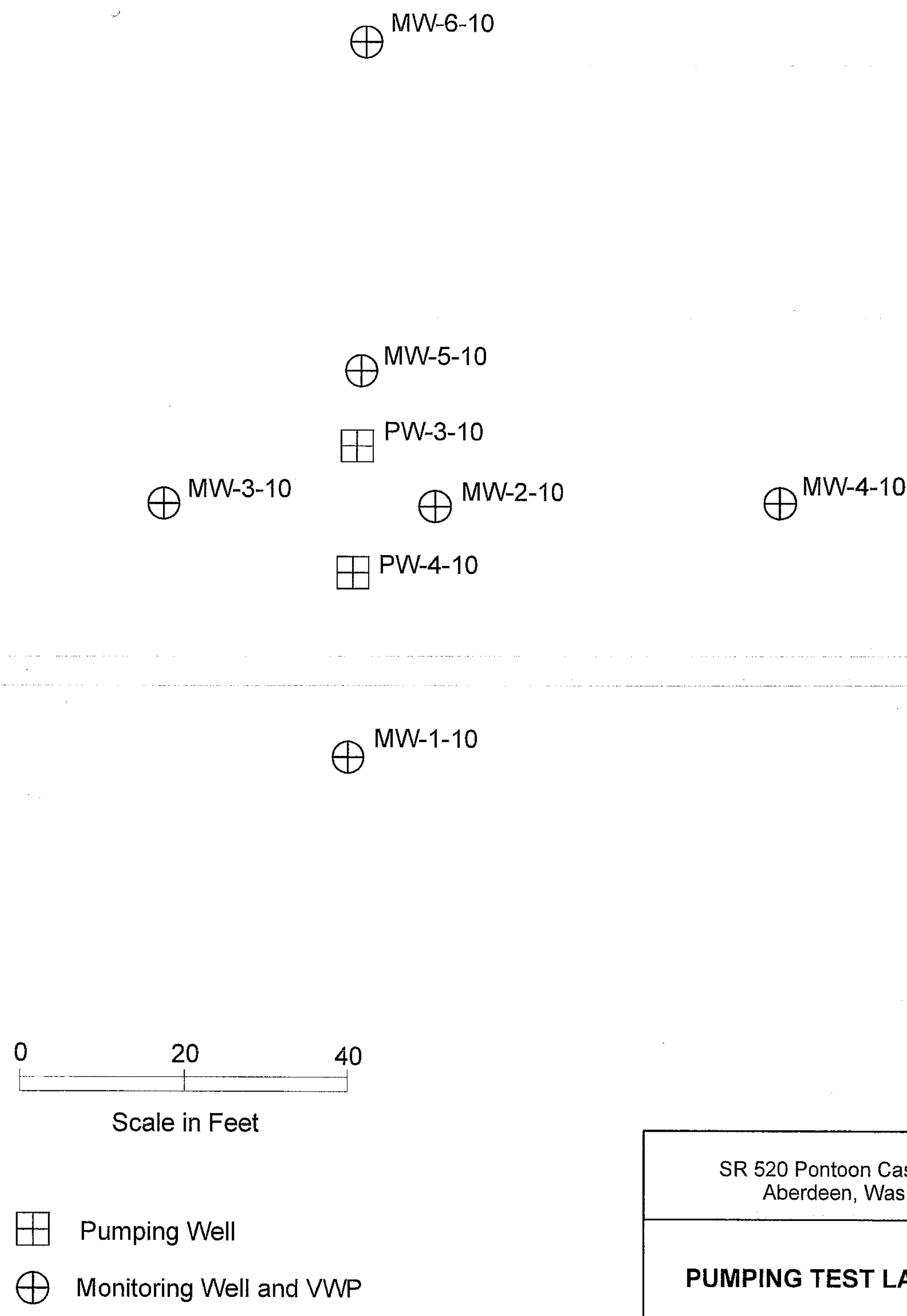
gpm = gallons per minute

TABLE H-6
GROUNDWATER MODELING RESULTS, HIGH CONDUCTIVITY

Flow Model Time Step	Time (days)	Number of Wells Pumping	Dewatering Well Discharge (gpm)	Cutoff Perimeter Trench Drain Discharge (gpm)	Total Discharge Wells and Trench Drain (gpm)
1	6	18	292	63	355
2	12	24	325	84	409
3	18	30	417	117	534
4	31	33	416	90	506
5	48		352	66	418
6	67		316	58	374
7	90		291	53	344
8	118		274	50	324

Note:

gpm = gallons per minute



SR 520 Pontoon Casting Facility
Aberdeen, Washington

PUMPING TEST LAYOUT PLAN

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
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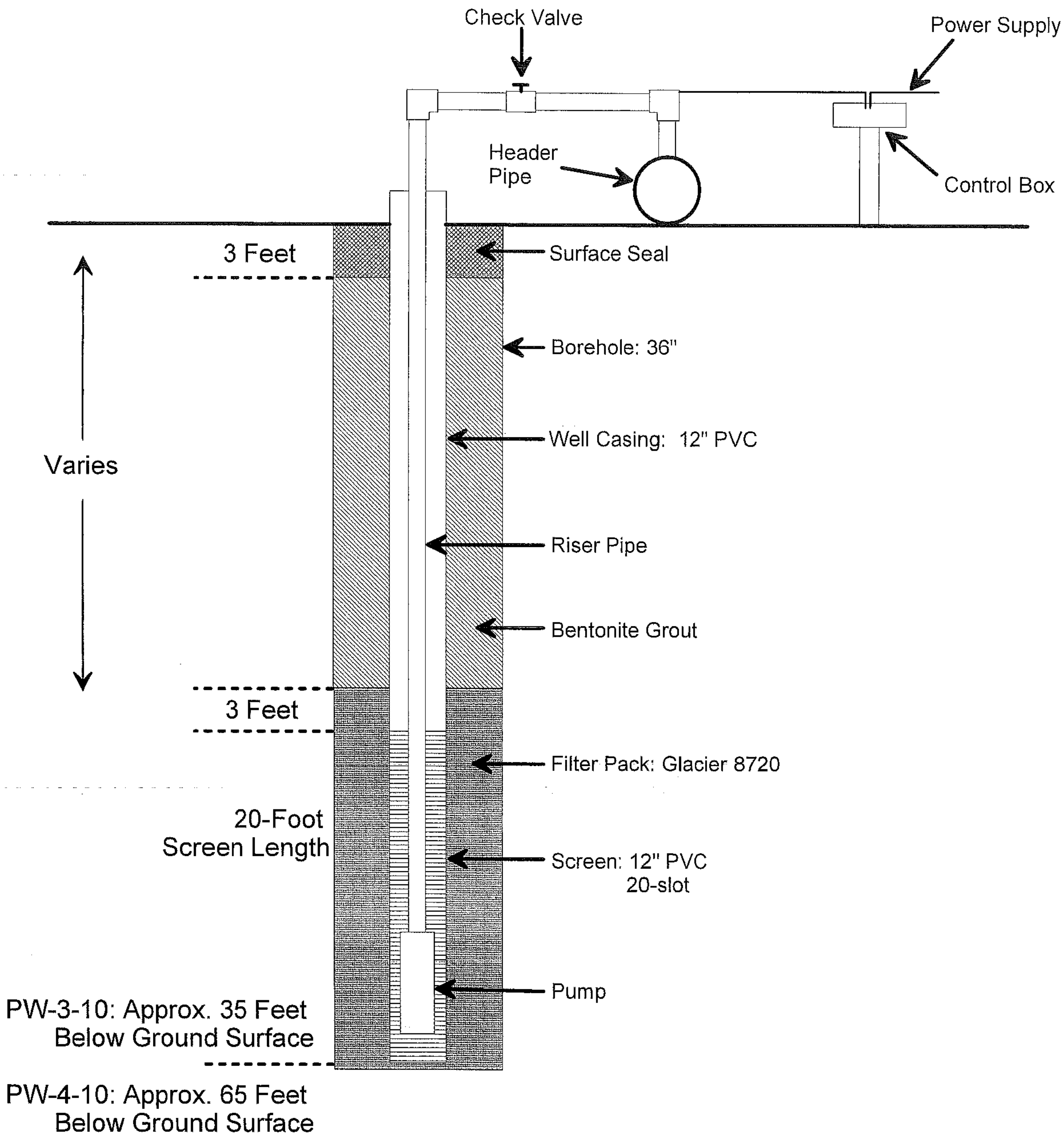
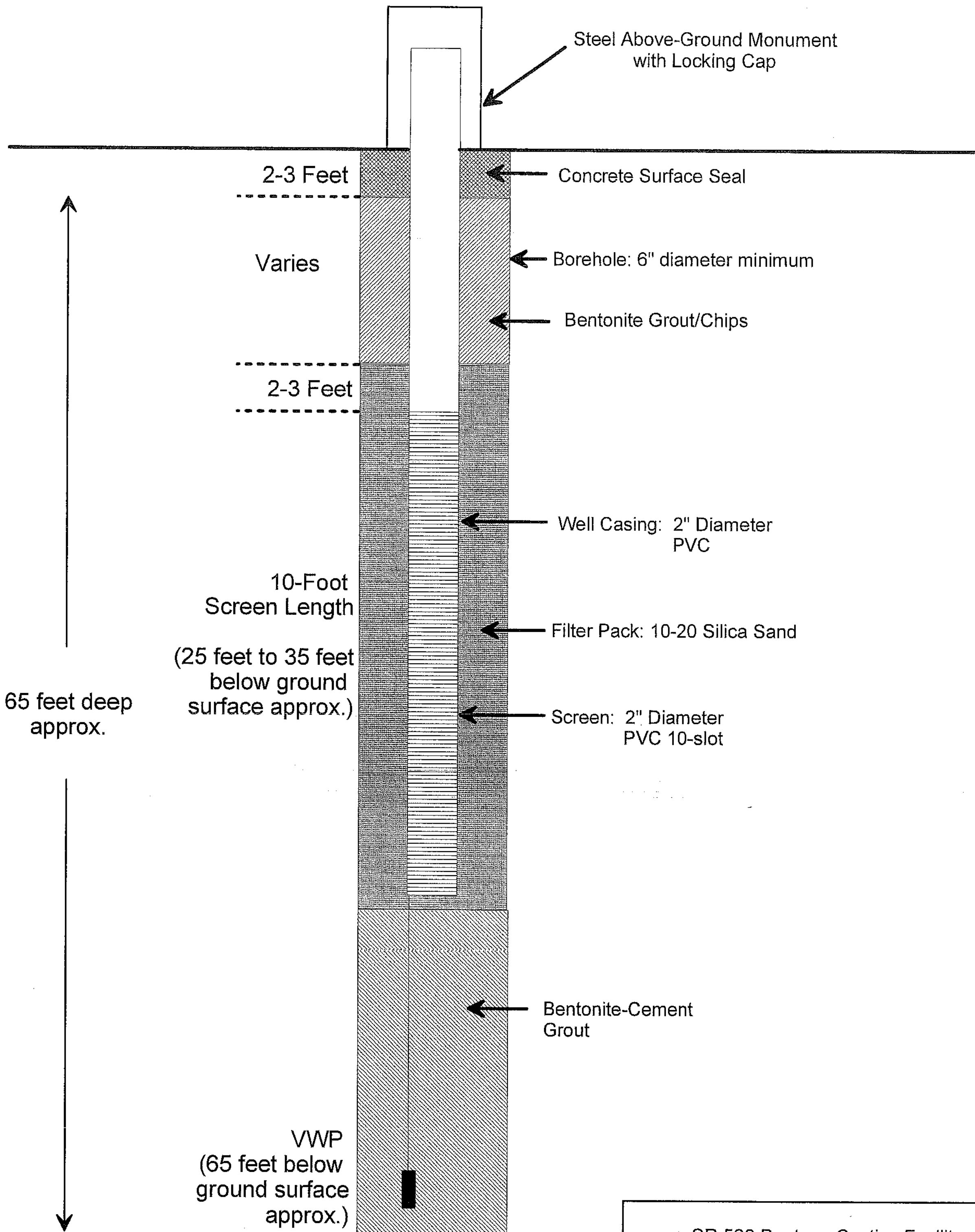
FIG. H-1



Not to Scale

SR 520 Pontoon Casting Facility Aberdeen, Washington	
TYPICAL PUMPING WELL SCHEMATIC PW-3-10 and PW-4-10	
September 2010	21-1-21190-014
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. H-2**



Not to Scale

SR 520 Pontoon Casting Facility
Aberdeen, Washington

TYPICAL MONITORING WELL AND VWP SCHEMATIC MW-1-10, MW-4-10, and MW-5-10

September 2010 21-1-21190-014

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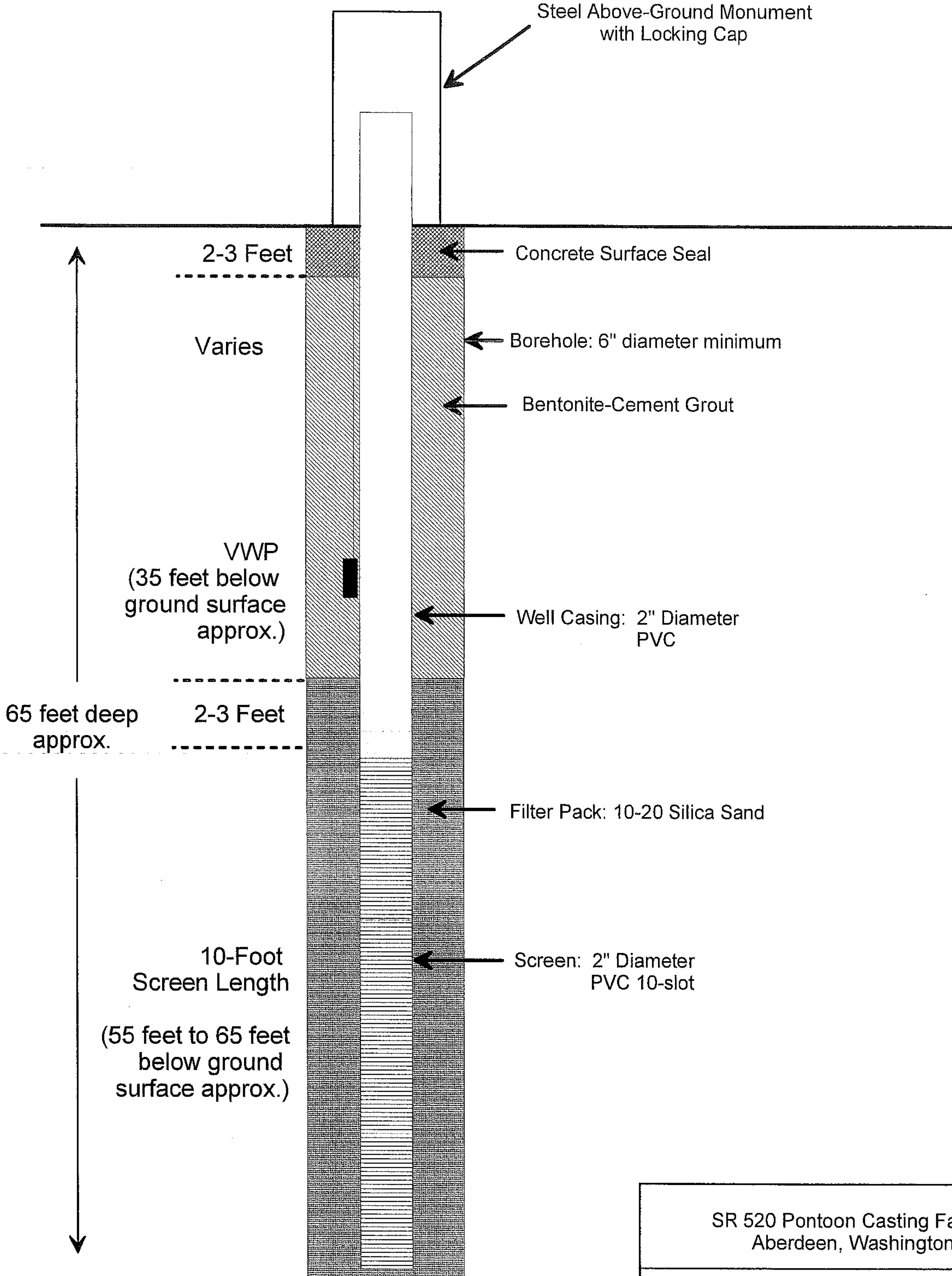
FIG. H-3



Not to Scale

SR 520 Pontoon Casting Facility
Aberdeen, Washington

TYPICAL MONITORING WELL AND VWP SCHEMATIC MW-2-10, MW-3-10, and MW-6-10

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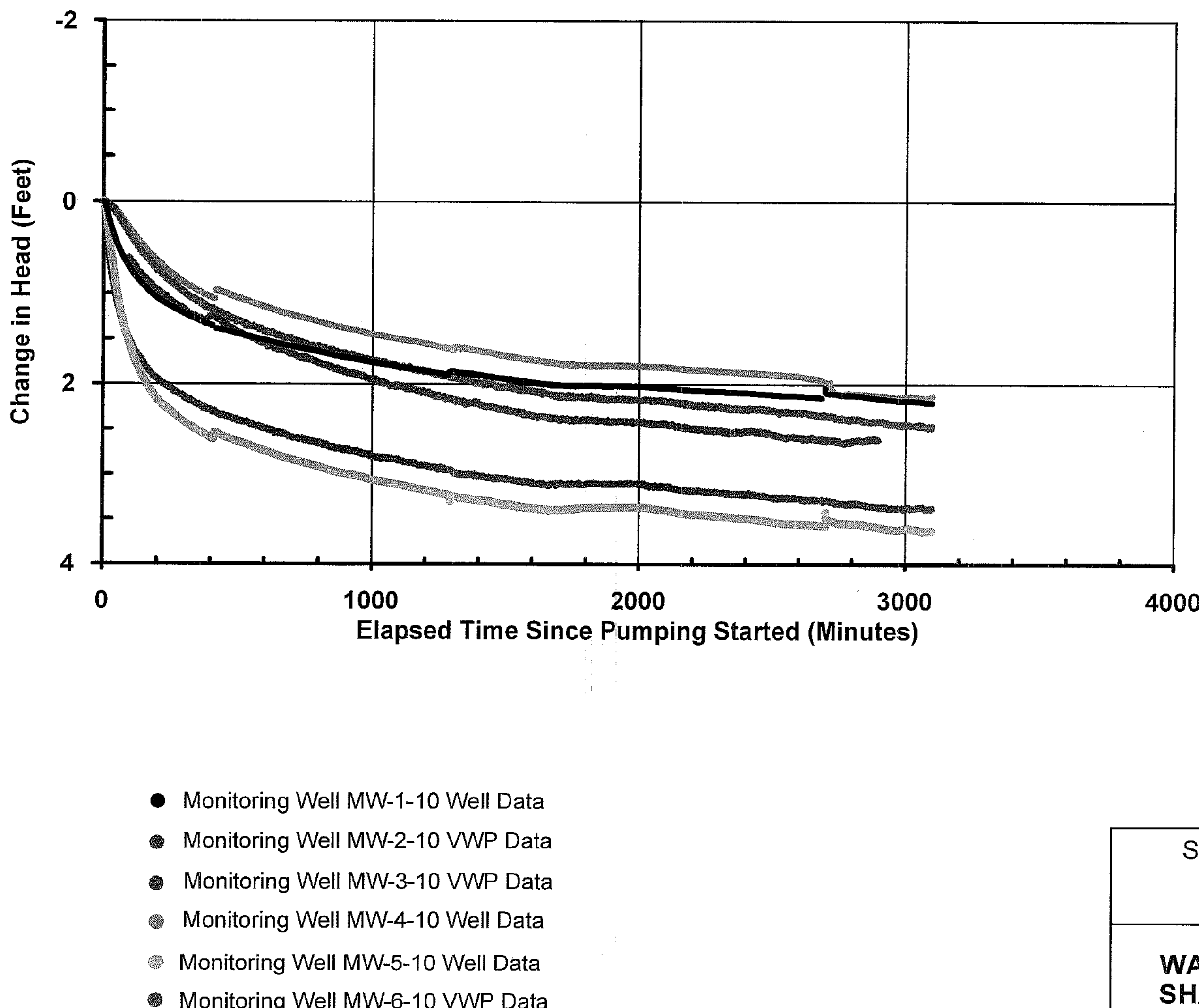
FIG. H-4



SR 520 Pontoon Casting Facility
Aberdeen, Washington

WATER LEVEL HYDROGRAPH
SHALLOW INSTRUMENTATION
PW-3-10 SHALLOW PUMPING TEST 1

September 2010 | 21-1-21190-014

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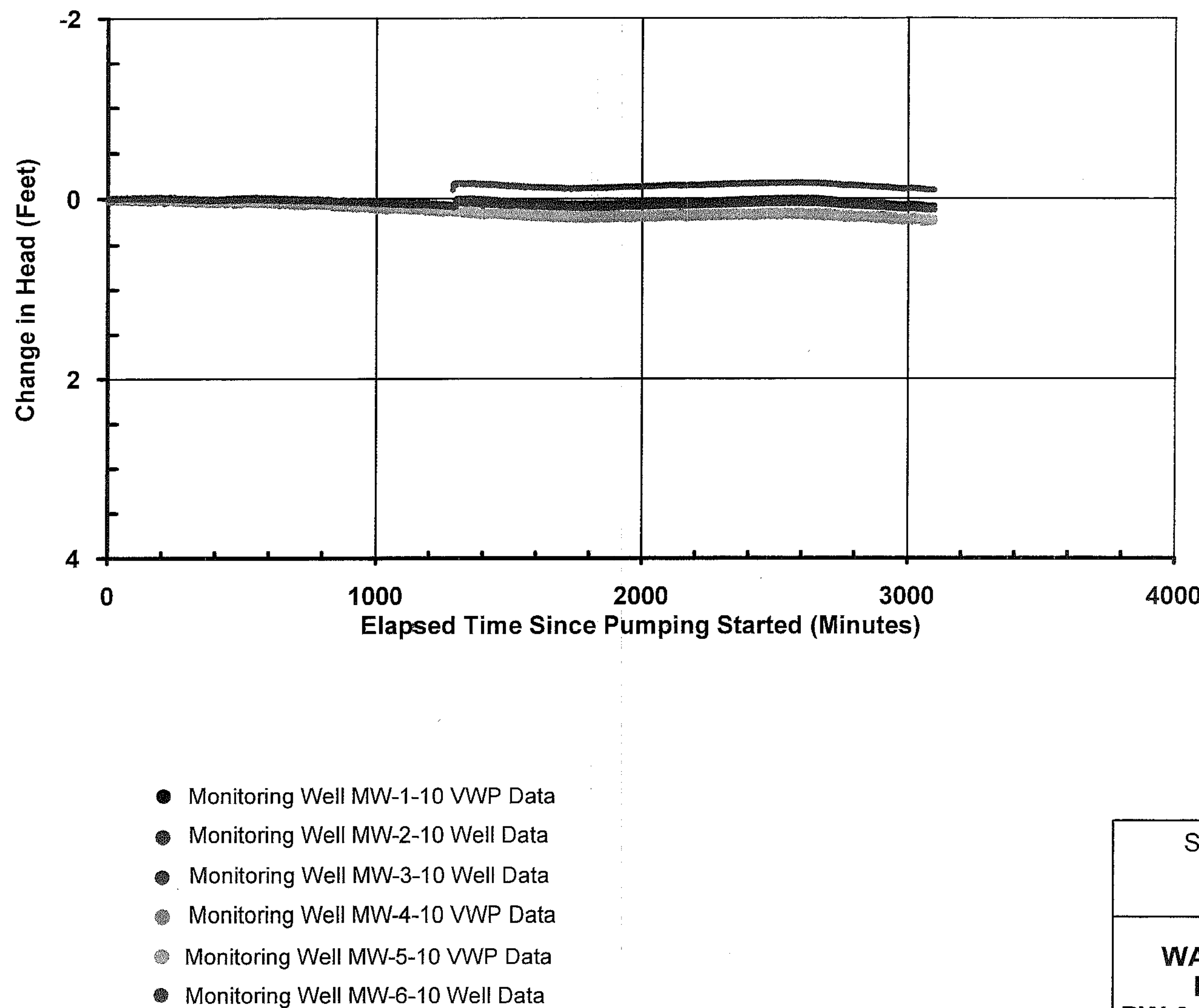
FIG.H-5.1

FIG.H-5.1



SR 520 Pontoon Casting Facility
Aberdeen, Washington

WATER LEVEL HYDROGRAPH DEEP INSTRUMENTATION PW-3-10 SHALLOW PUMPING TEST 1

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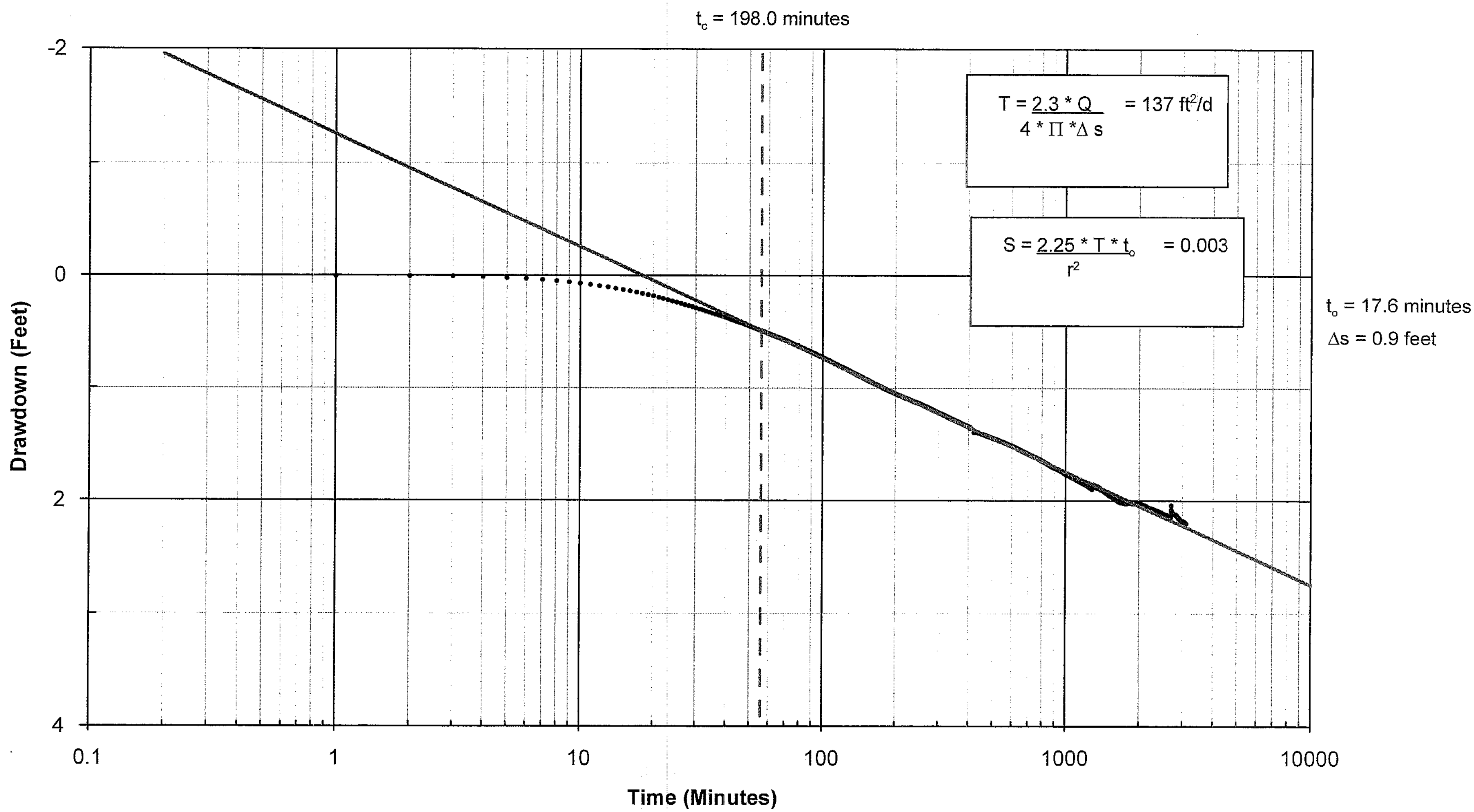
FIG.H-5.2

FIG.H-5.2



NOTE: See Report for discussion of Cooper-Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-1-10 SHALLOW WELL
COOPER-JACOB ANALYSIS

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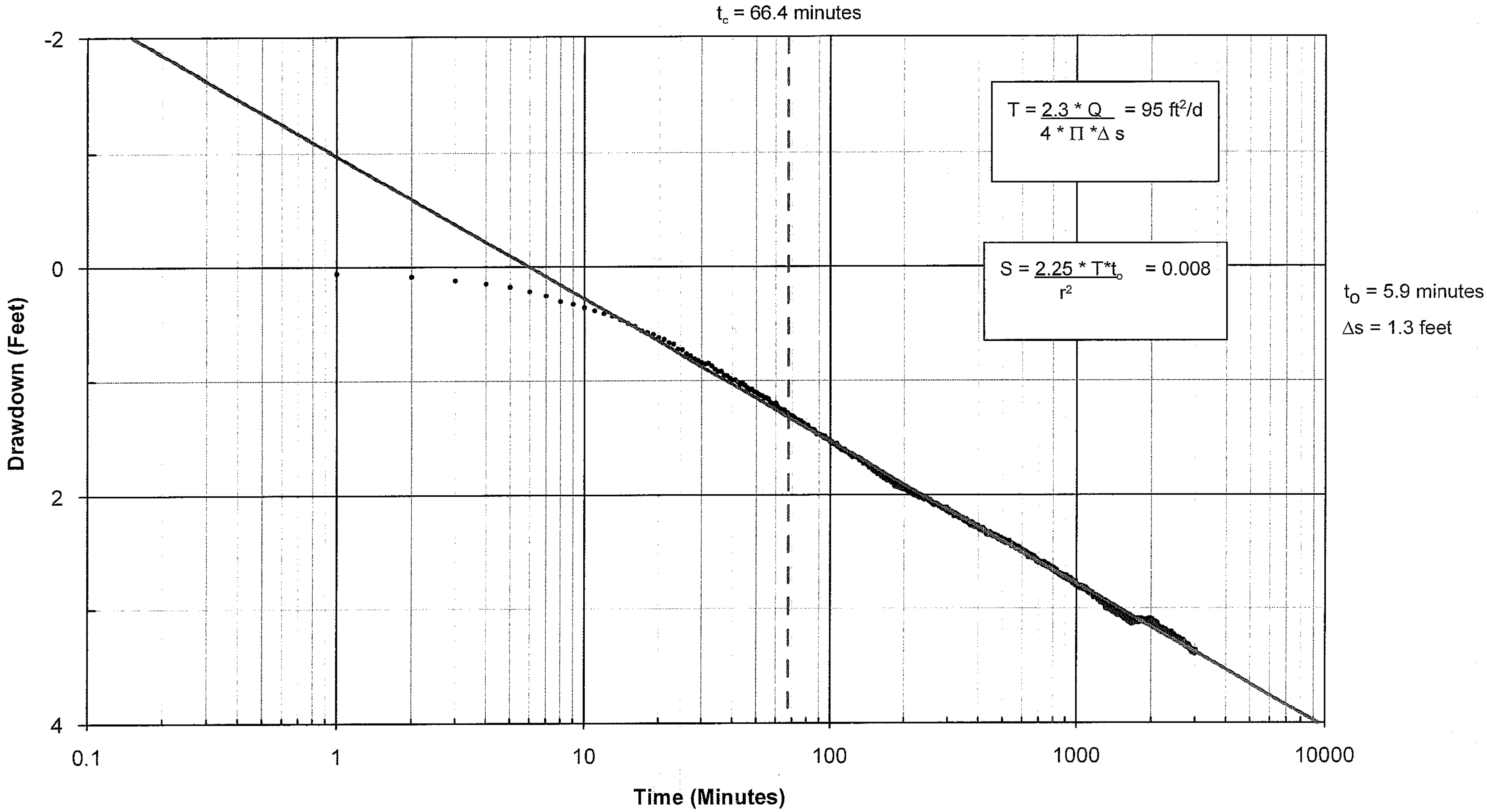
FIG. H-5.3

FIG. H-5.3



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-2-10 SHALLOW VWP
COOPER-JACOB ANALYSIS

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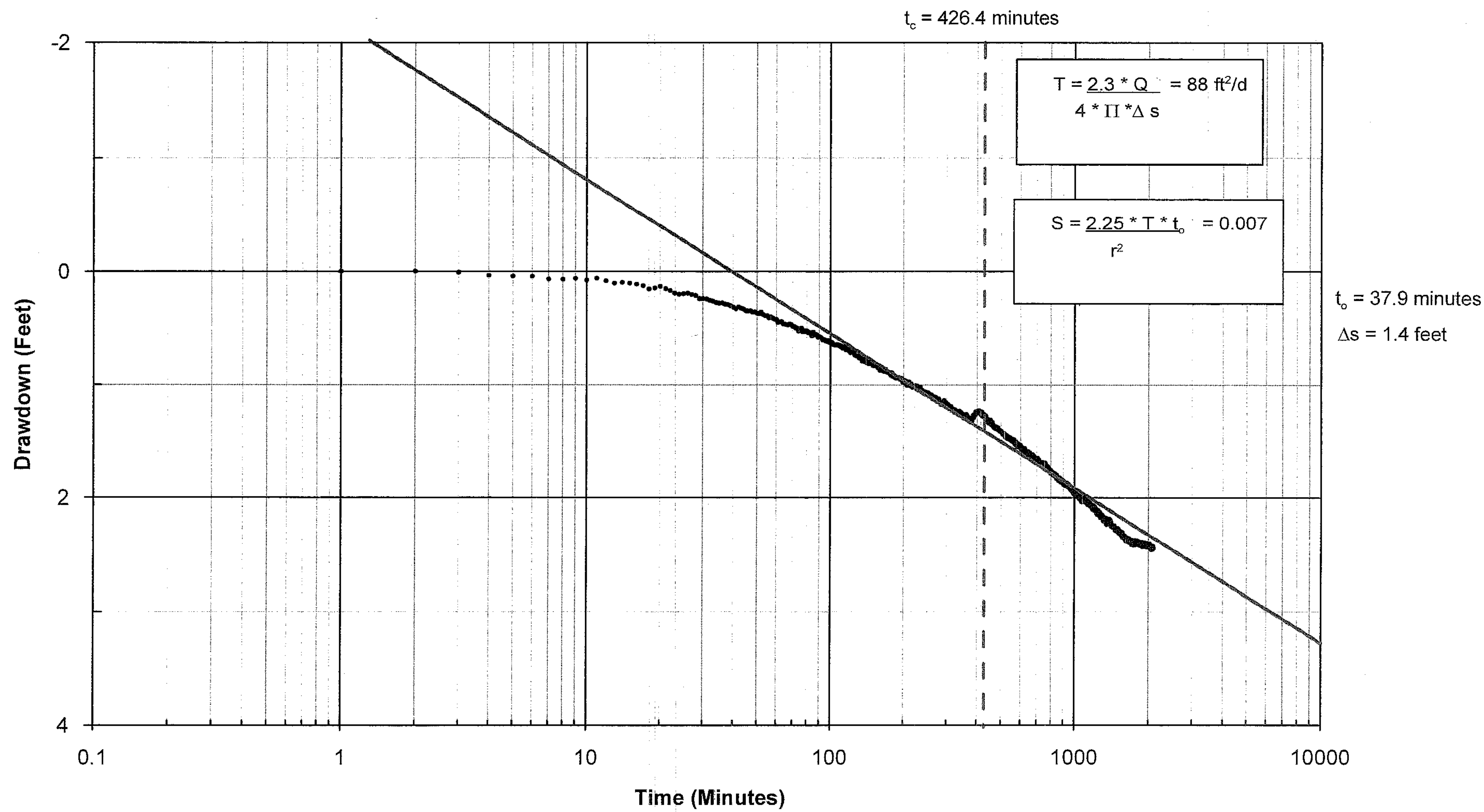
FIG. H-5.4

FIG. H-5.4



NOTE: See Report for discussion of the Cooper-jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-3-10 SHALLOW VWP
COOPER-JACOB ANALYSIS

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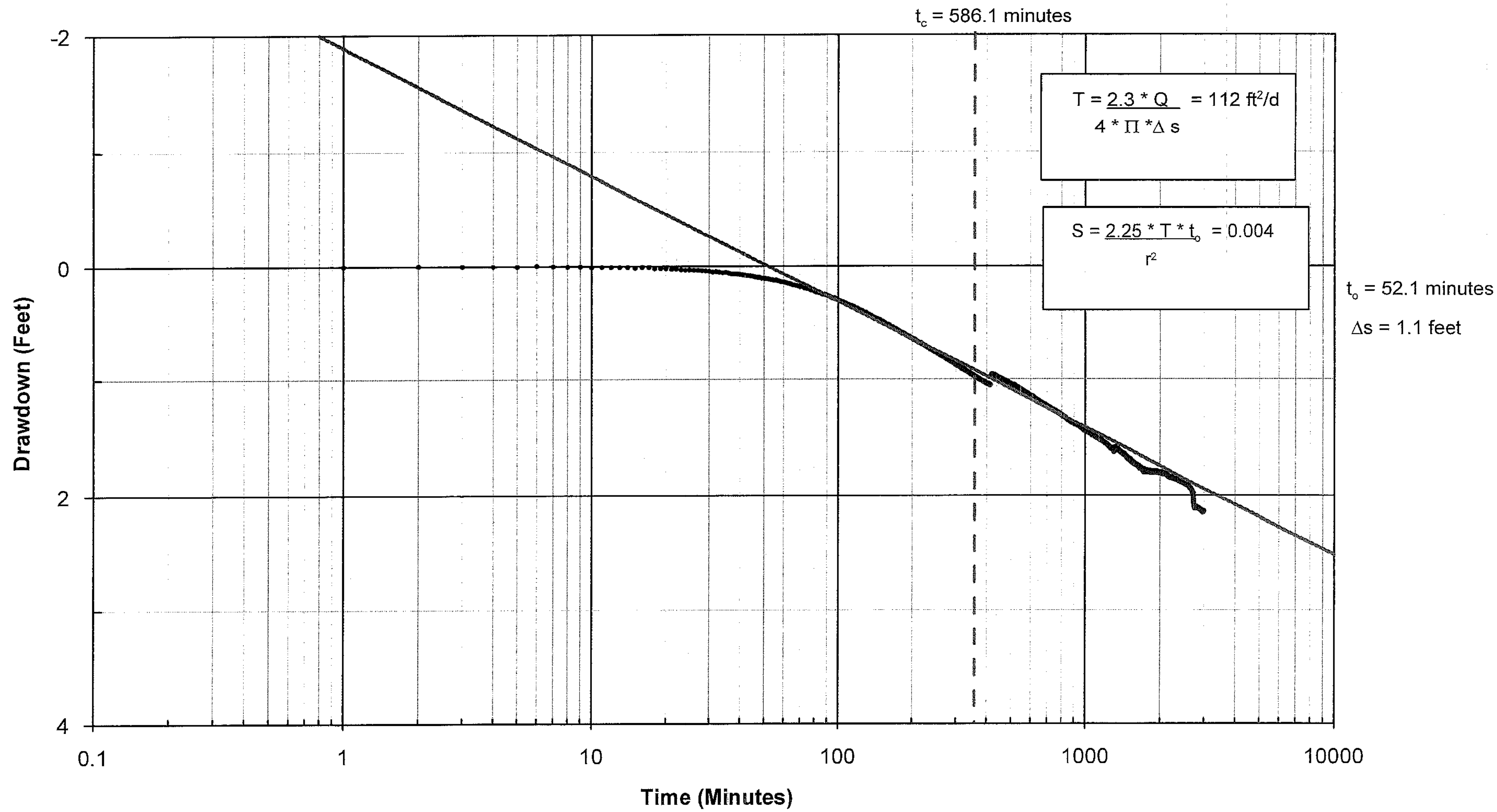
FIG.H-5.5

FIG. H-5.5



NOTE: See report for discussion of the Cooper-Jacob Straigh Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-4-10 SHALLOW WELL
COOPER-JACOB ANALYSIS

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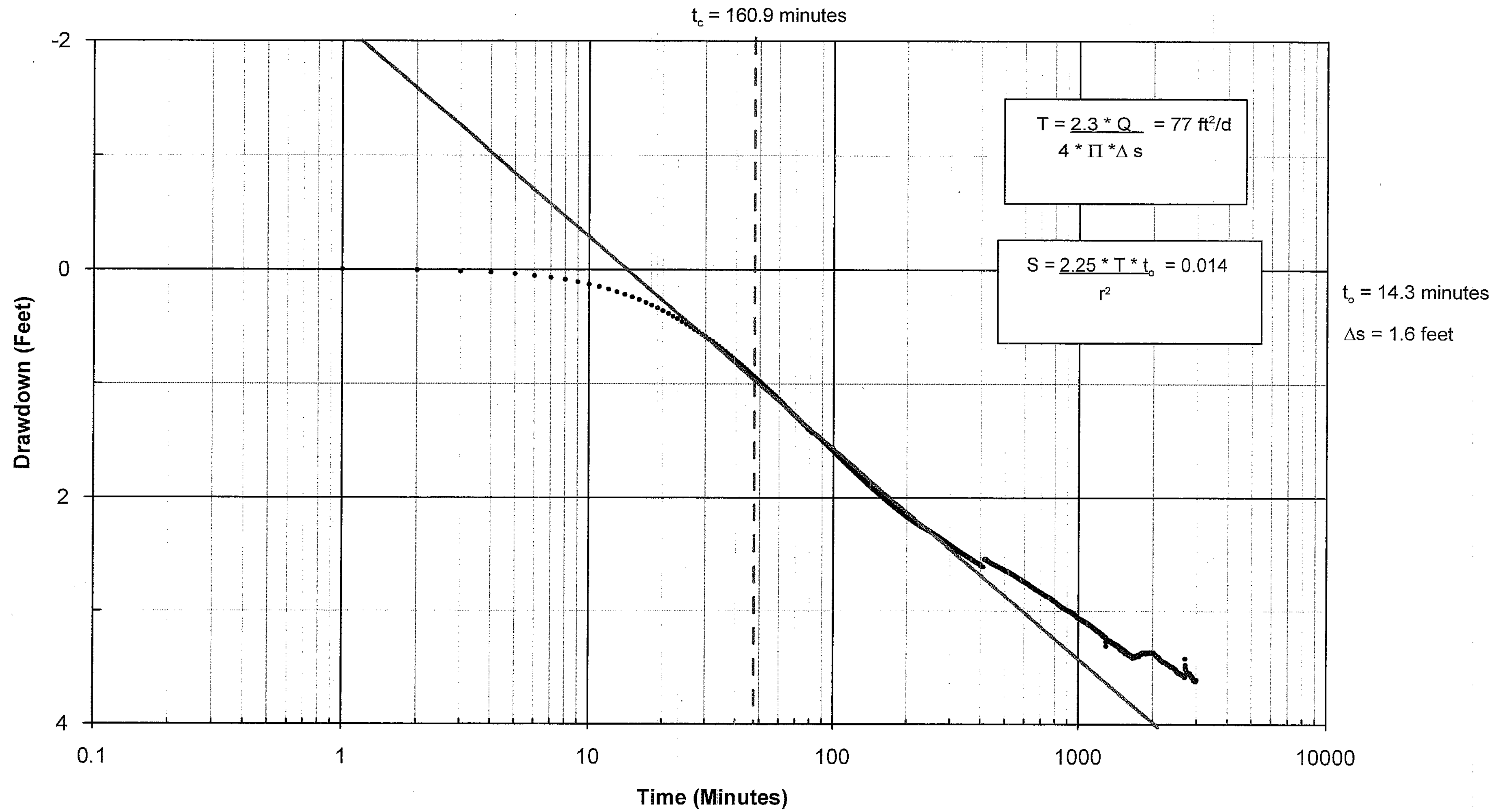
FIG. H-5.6

FIG. H-5.6



NOTE: See Report for disccusion of the Cooper-Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-5-10 SHALLOW WELL
COOPER-JACOB ANALYSIS

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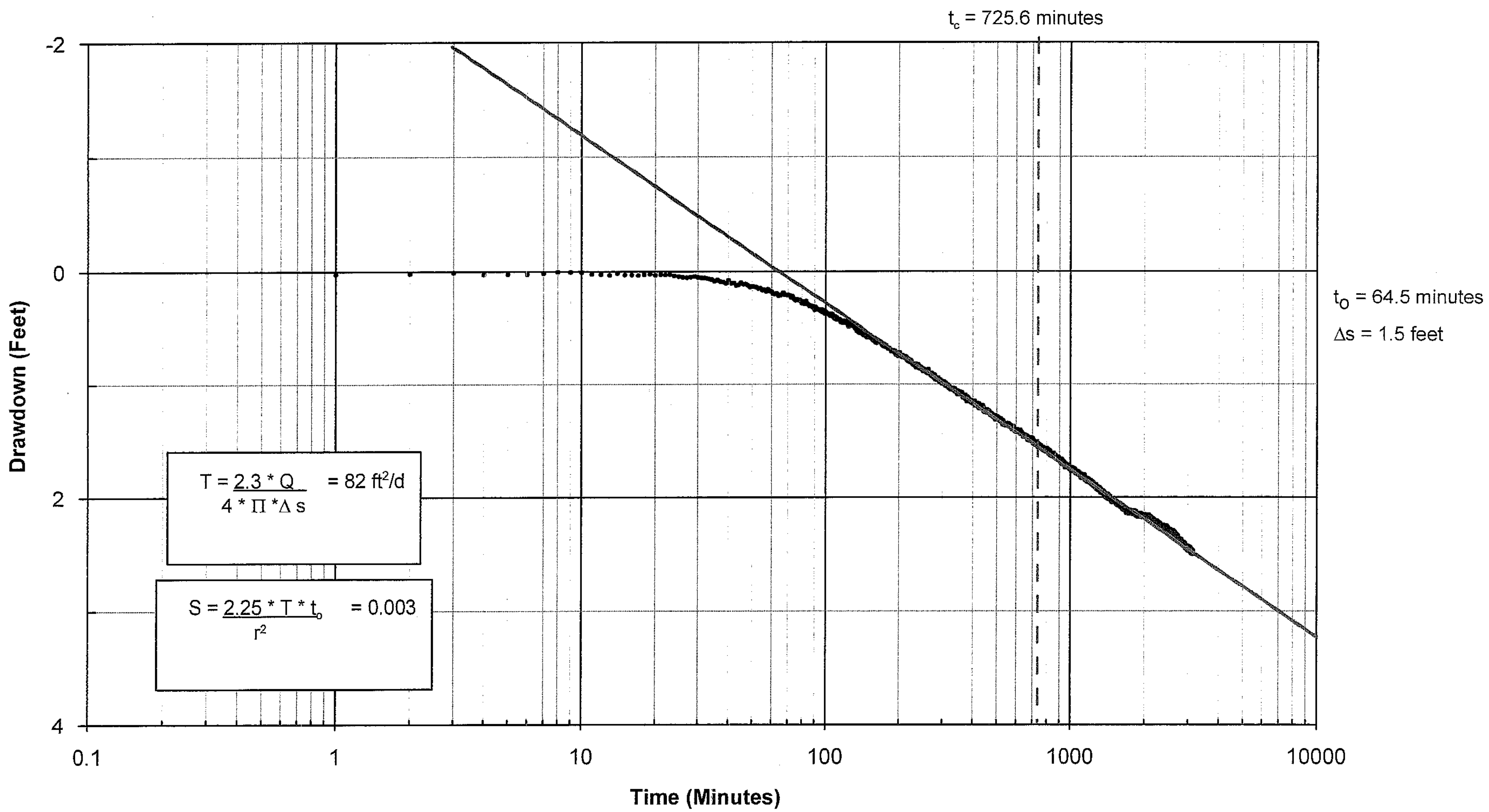
FIG. H-5.7

FIG. H-5.7



NOTE: See Report for discussion of the Cooper-Jacob Straight line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-6-10 SHALLOW VWP
COOPER-JACOB ANALYSIS

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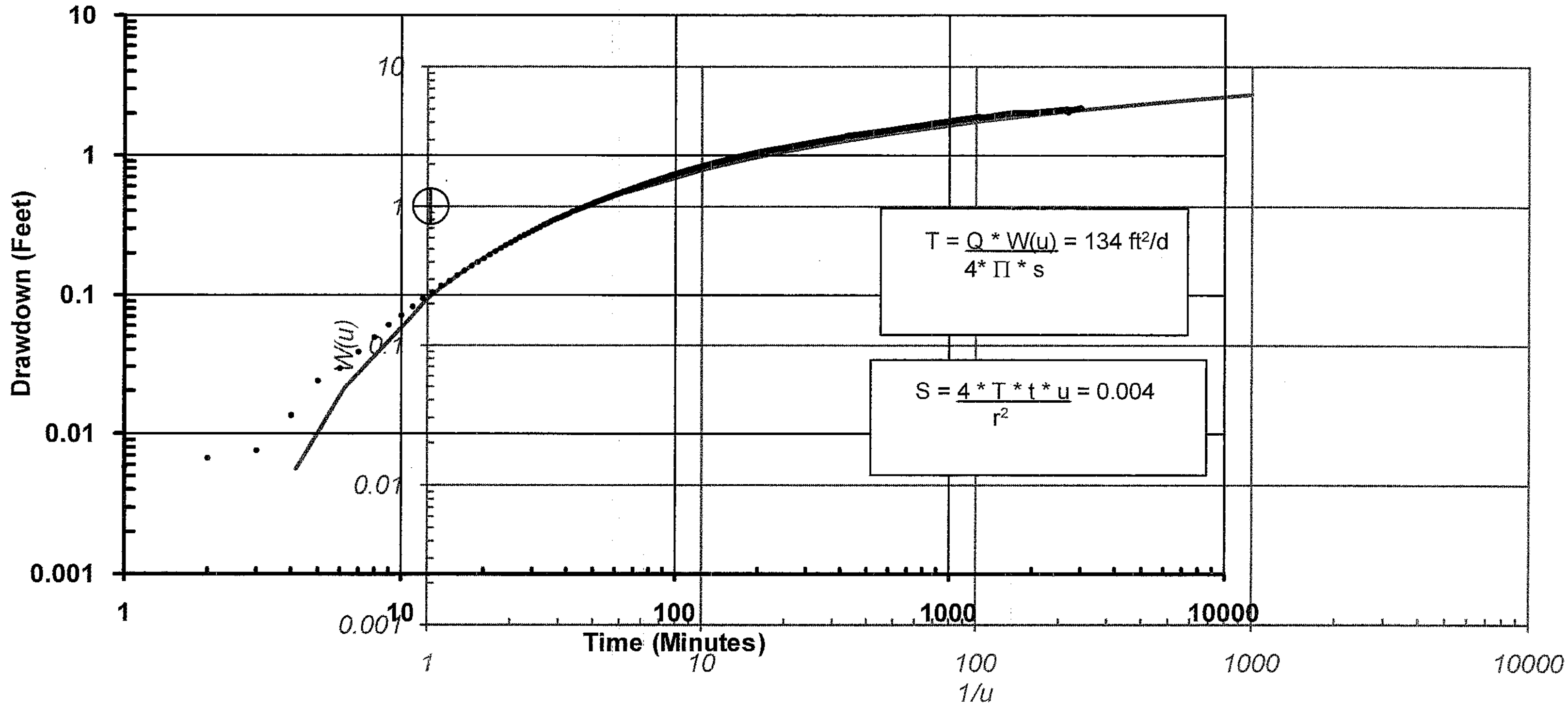
FIG. H-5.8

FIG. H-5.8



● Monitoring Well MW-1-10 Shallow Drawdown Data

— Theis Curve

⊕ Match Point

NOTE: See Report for the discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point
W(u) = 1
1/u = 1
t = 12.6 min
s = 0.4 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-1-10 SHALLOW WELL
THEIS CURVE MATCH

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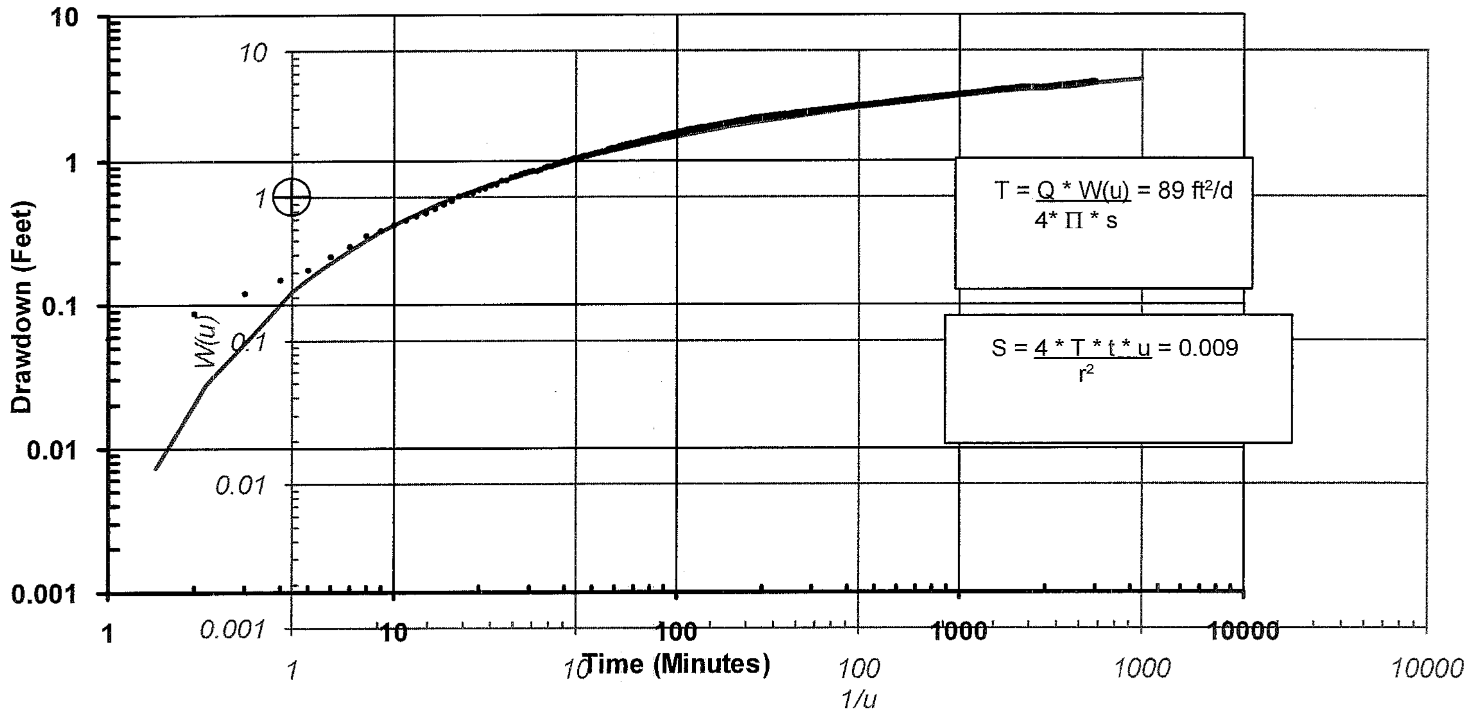
FIG. H-5.9

FIG. H-5.9



● Monitoring Well MW-2-10 Shallow VWP Drawdown Data

— Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 4.3 min

s = 0.6 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-2-10 SHALLOW VWP
THEIS CURVE MATCH

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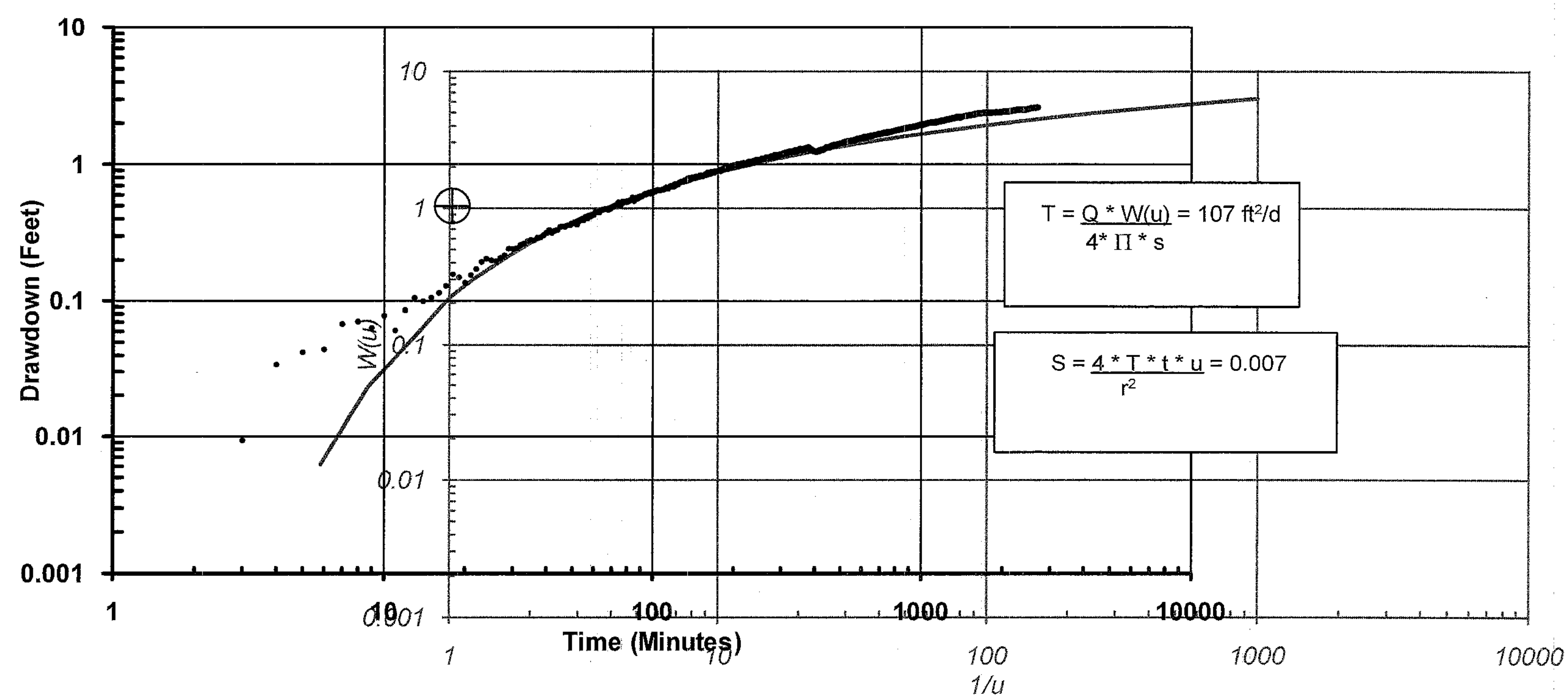
FIG. H-5.10

FIG. H-5.10



$T = \frac{Q * W(u)}{4 * \Pi * s} = 107\ ft^2/d$

$S = \frac{4 * T * t * u}{r^2} = 0.007$

● Monitoring Well MW-3-10 Shallow VWP Drawdown Data

—— Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 17.8 min

s = 0.5 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-3-10 SHALLOW VWP
THEIS CURVE MATCH

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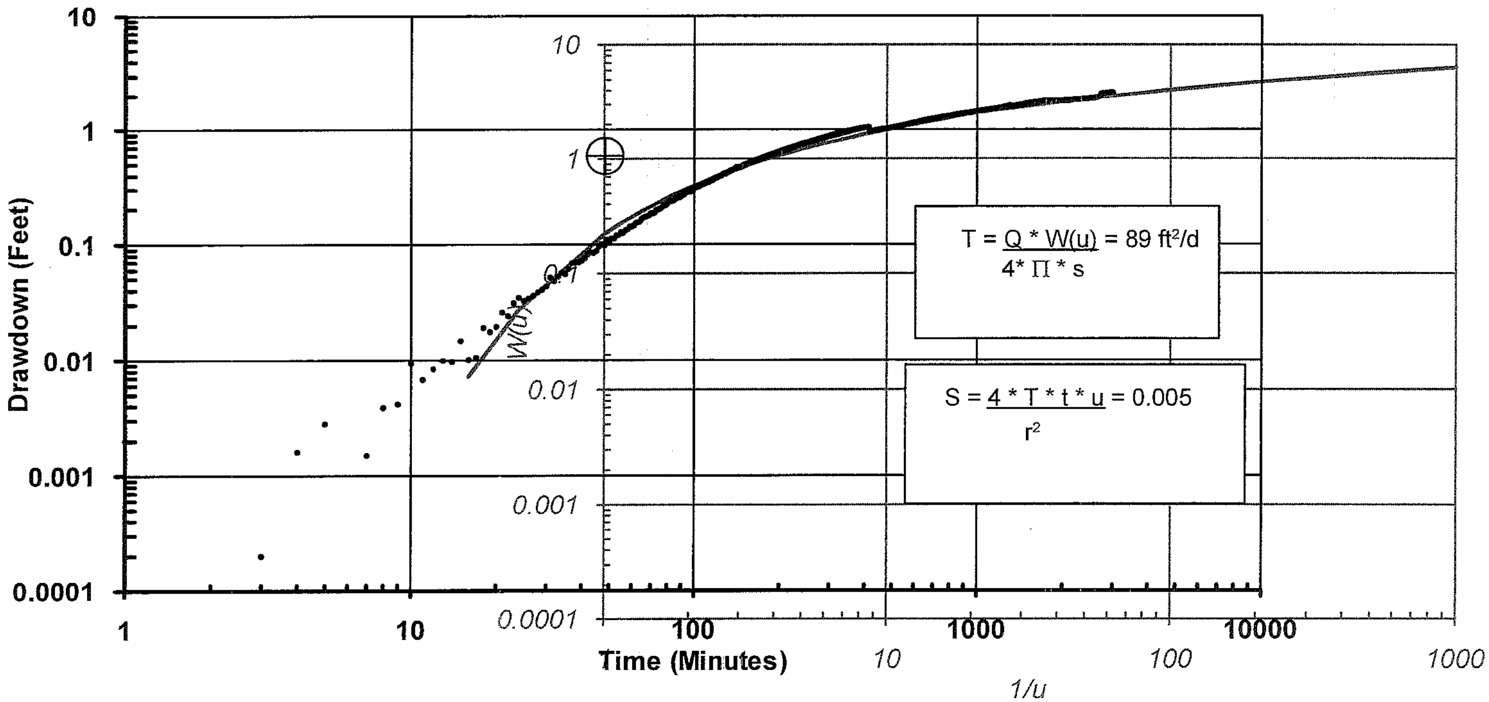
FIG. H-5.11

FIG. H-5.11



● Monitoring Well MW-4-10 Shallow Drawdown Data

— Theis Curve

⊕ Match Point

Match Point

W(u) = 1

1/u = 1

t = 48.3 min

s = 0.6 ft

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-4-10 SHALLOW WELL
THEIS CURVE MATCH

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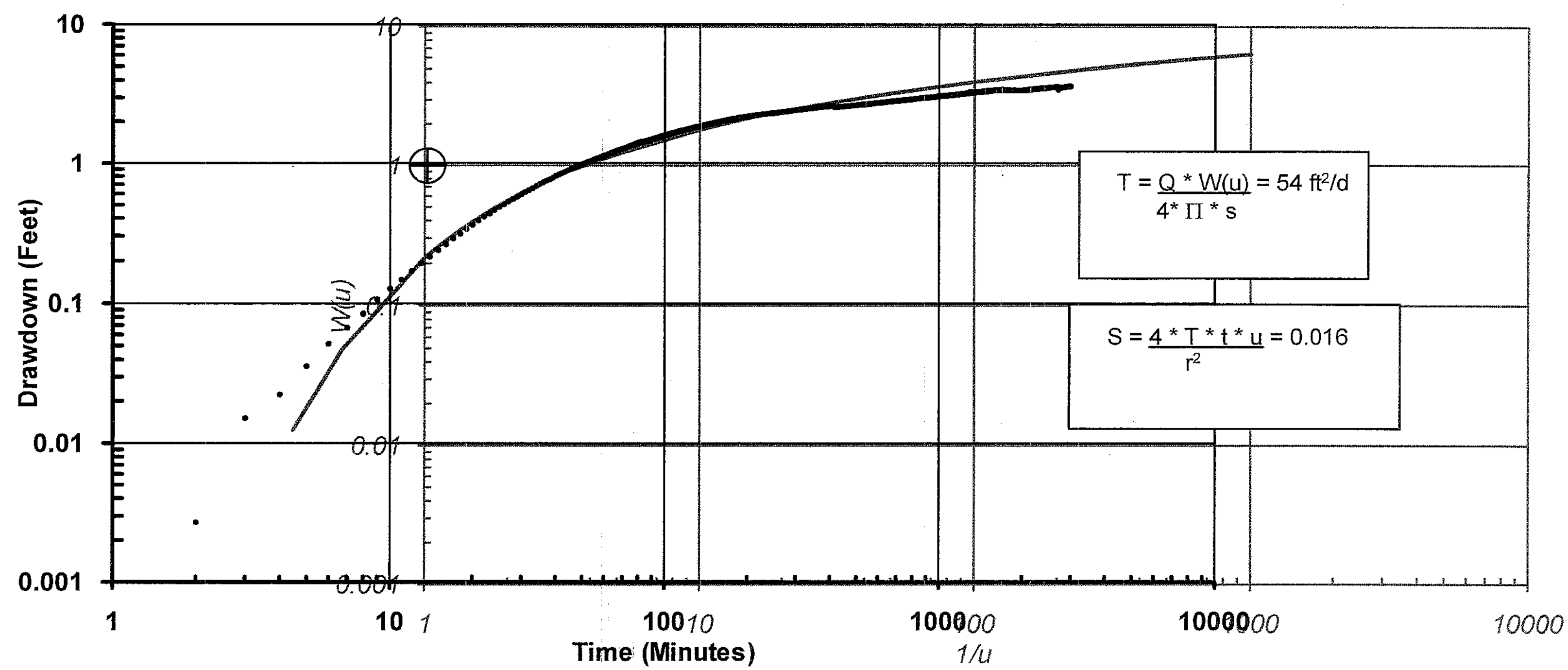
FIG. H-5.12

FIG. H-5.12



● Monitoring Well MW-5-10 Shallow Drawdown Data

— Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 13.6 min

s = 1.0 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-5-10 SHALLOW WELL
THEIS CURVE MATCH

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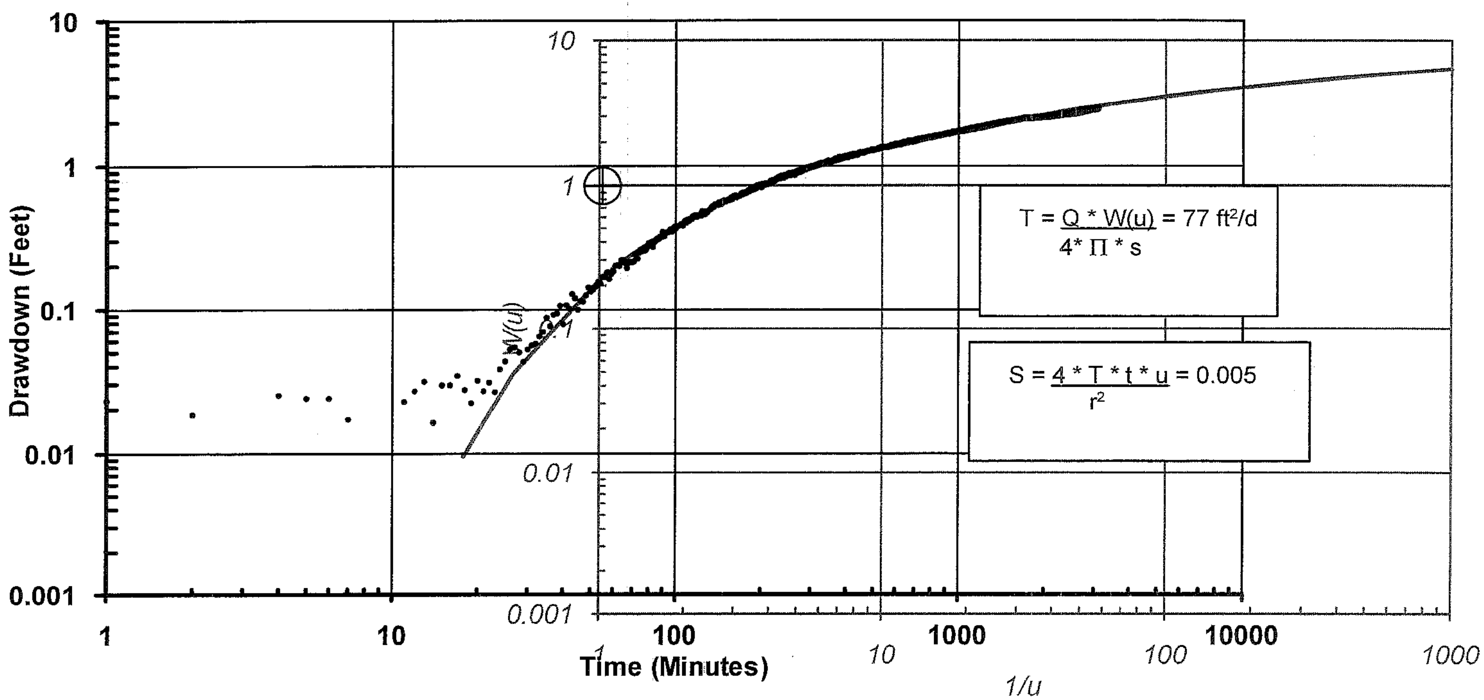
FIG. H-5.13

FIG. H-5.13



$$T = \frac{Q * W(u)}{4 * \Pi * s} = 77 \text{ ft}^2/\text{d}$$

$$S = \frac{4 * T * t * u}{r^2} = 0.005$$

● Monitoring Well MW-6-10 Shallow VWP Drawdown Data

— Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 55.0 min

s = 0.7 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-6-10 SHALLOW VWP
THEIS CURVE MATCH

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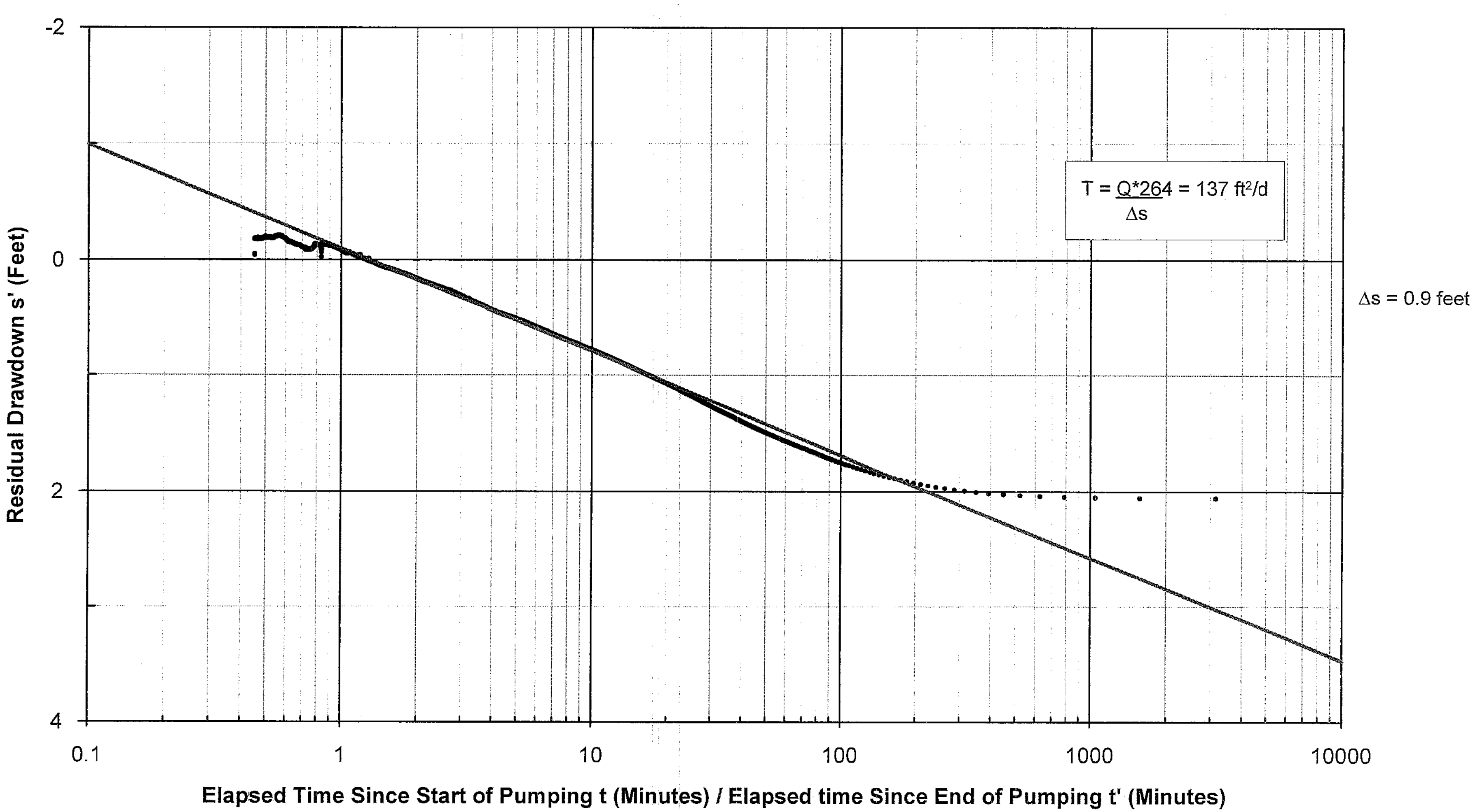
FIG. H-5.14

FIG. H-5.14



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-1-10 SHALLOW WELL
RECOVERY DATA

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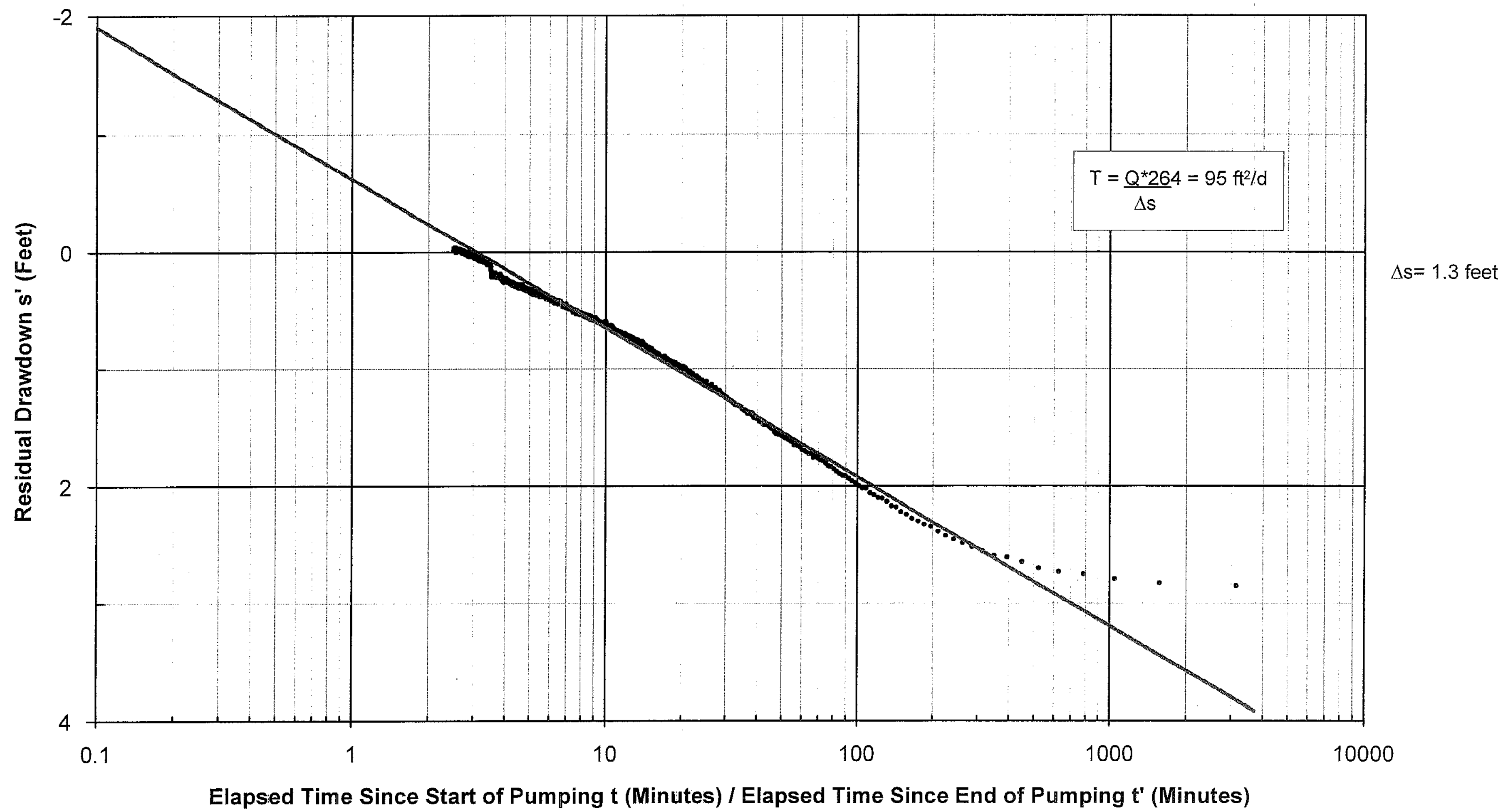
FIG. H-5.15

FIG. H-5.15



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-2-10 SHALLOW VWP
RECOVERY DATA

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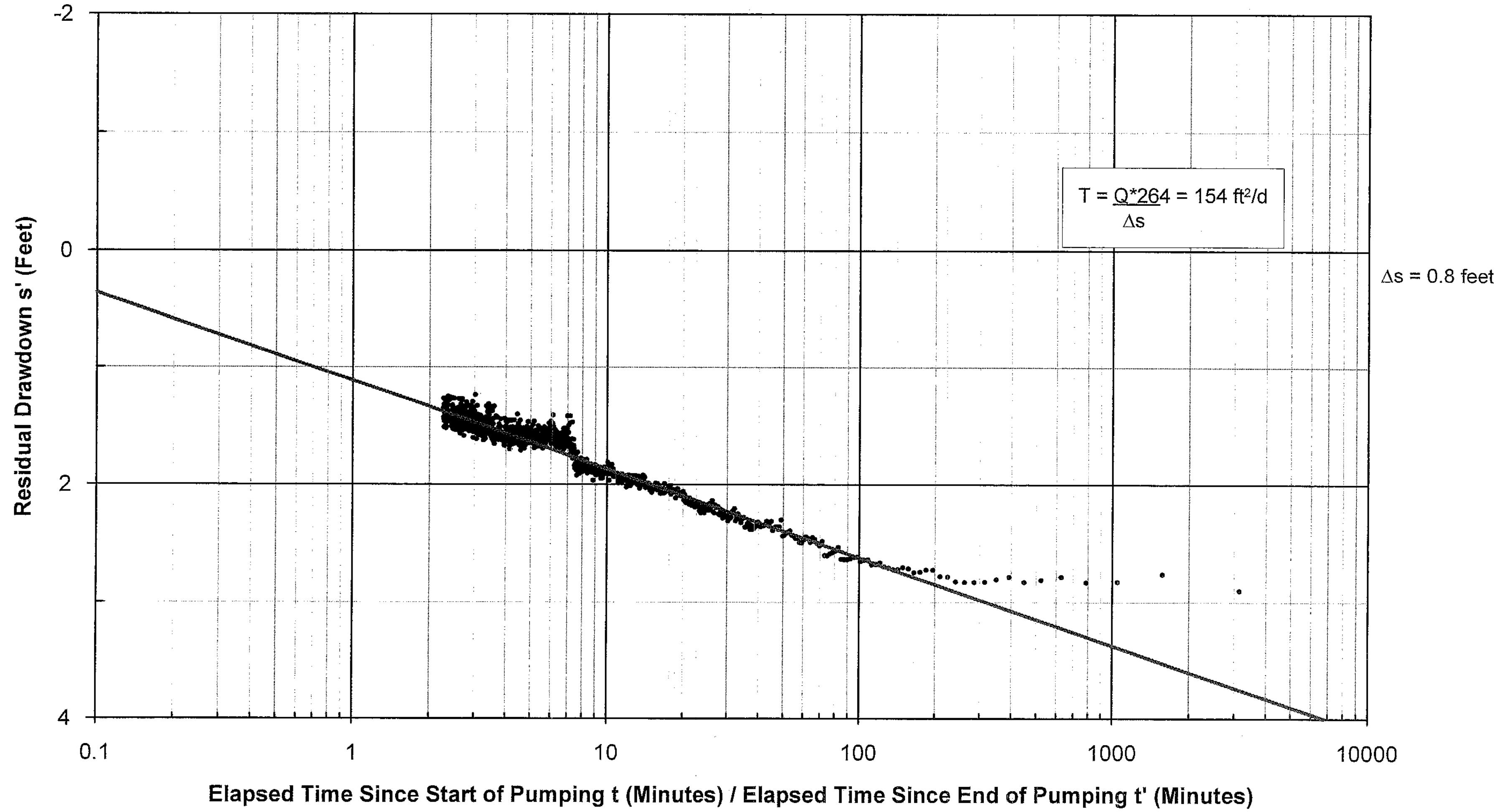
FIG. H-5.16

FIG. H-5.16



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-3-10 SHALLOW VWP
RECOVERY DATA

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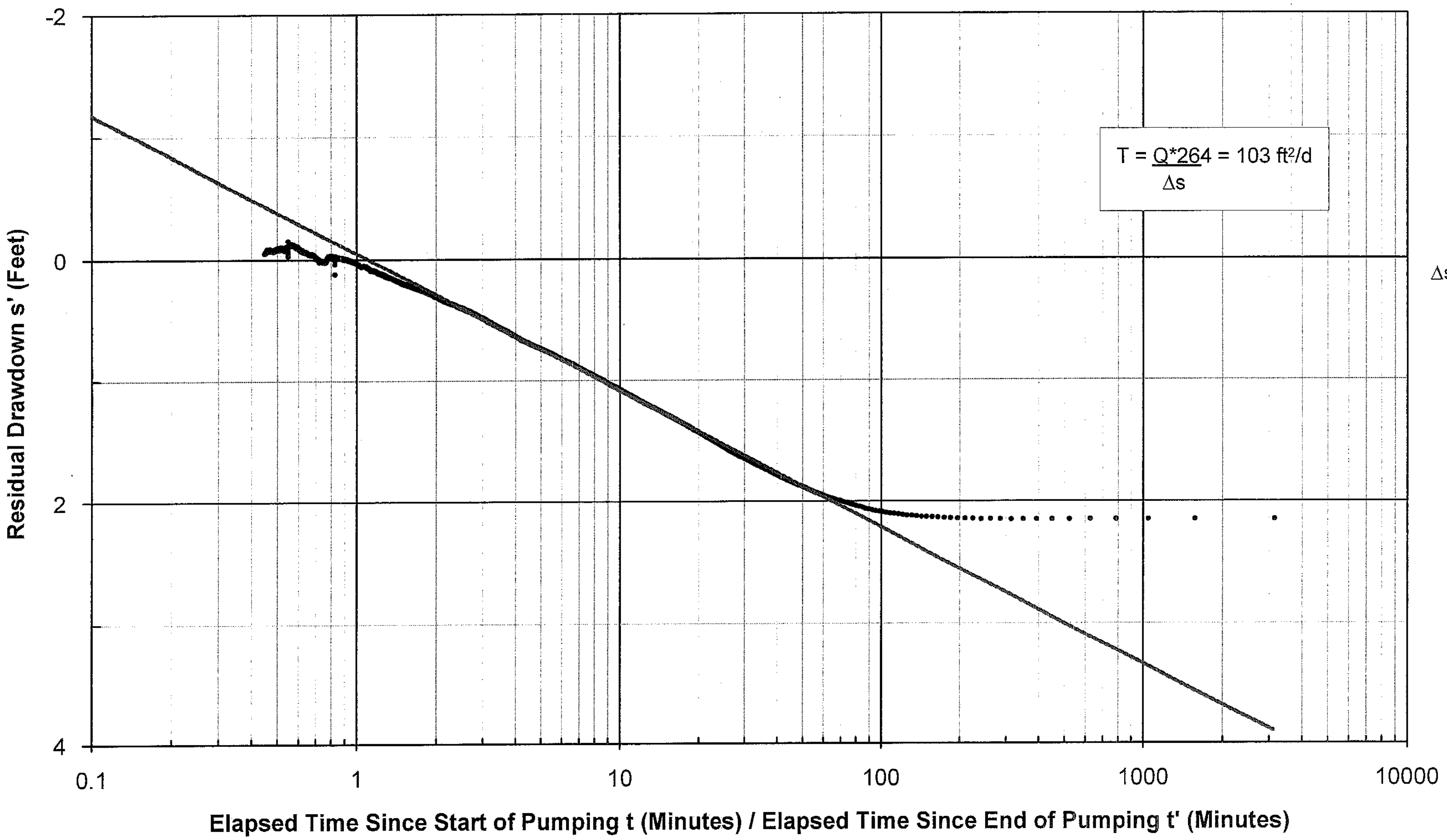
FIG. H-5.17

FIG. H-5.17



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-4-10 SHALLOW WELL
RECOVERY DATA

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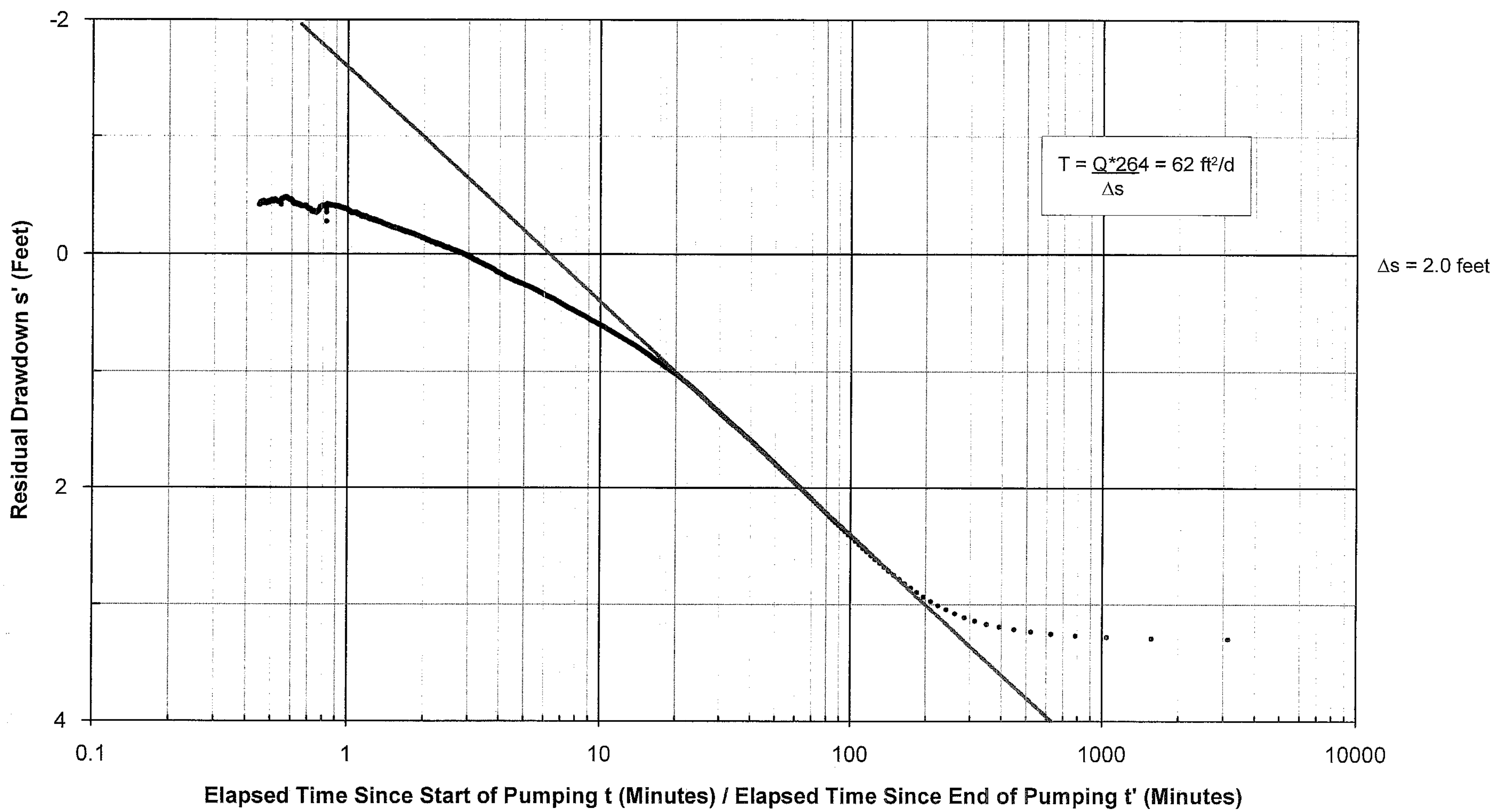
FIG. H-5.18

FIG. H-5.18



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-5-10 SHALLOW WELL
RECOVERY DATA

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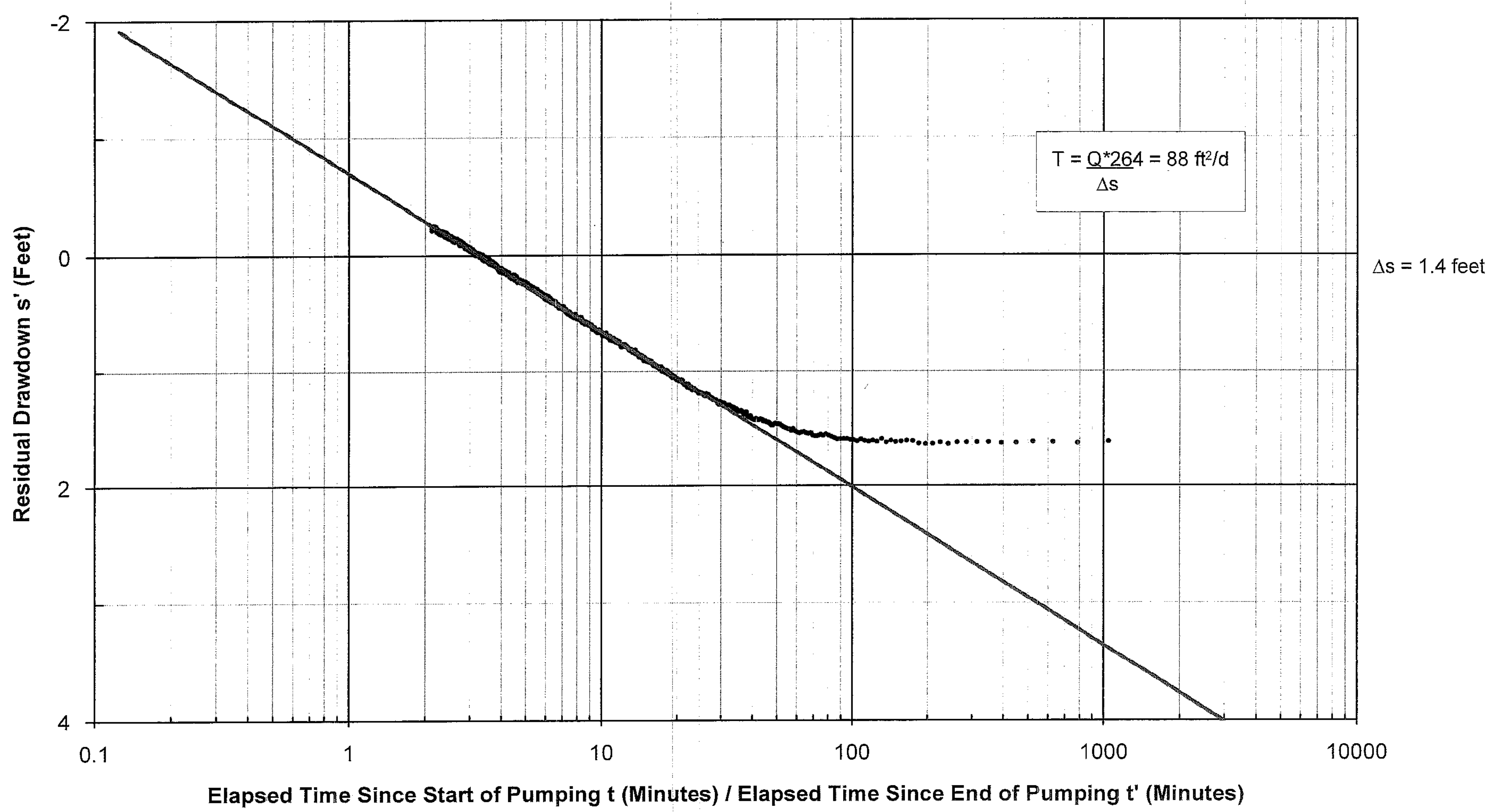
FIG. H-5.19

FIG. H-5.19



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 1
MW-6-10 SHALLOW VWP
RECOVERY DATA

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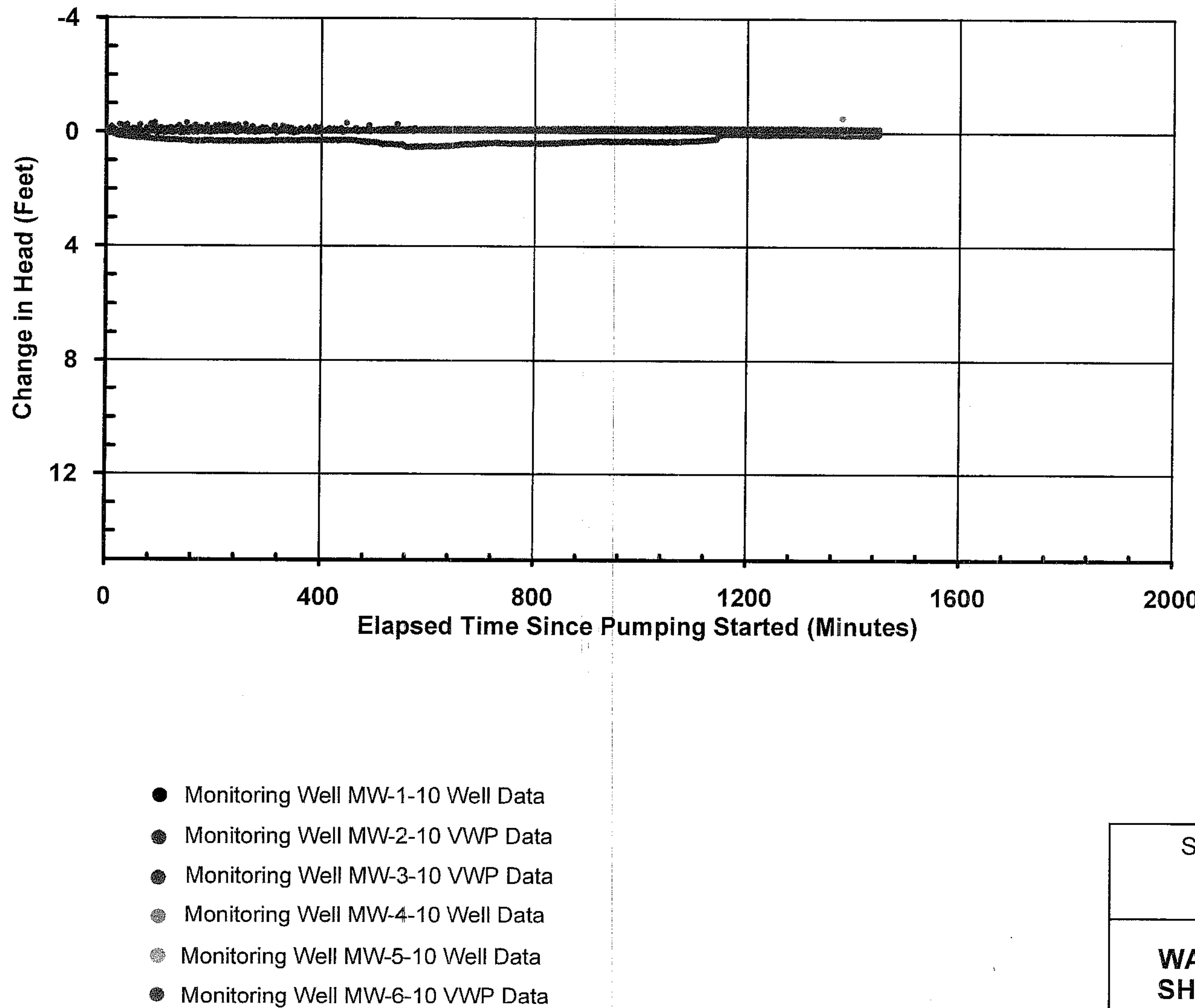
FIG. H-5.20

FIG. H-5.20



SR 520 Pontoon Casting Facility
Aberdeen, Washington

WATER LEVEL HYDROGRAPH
SHALLOW INSTRUMENTATION
PW-4-10 DEEP PUMPING TEST

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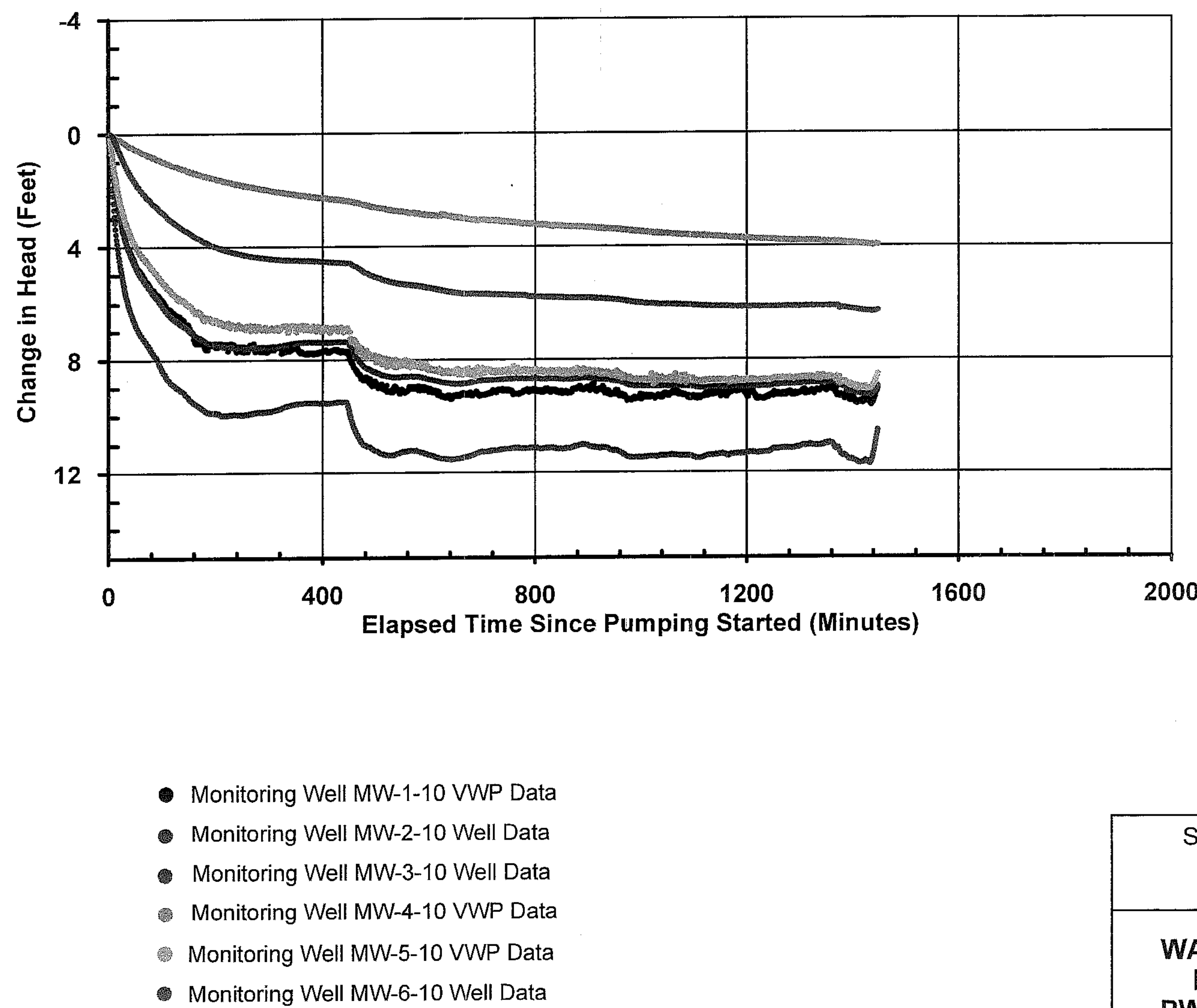
FIG.H-6.1

FIG.H-6.1



SR 520 Pontoon Casting Facility
Aberdeen, Washington

WATER LEVEL HYDROGRAPH
DEEP INSTRUMENTATION
PW-4-10 DEEP PUMPING TEST

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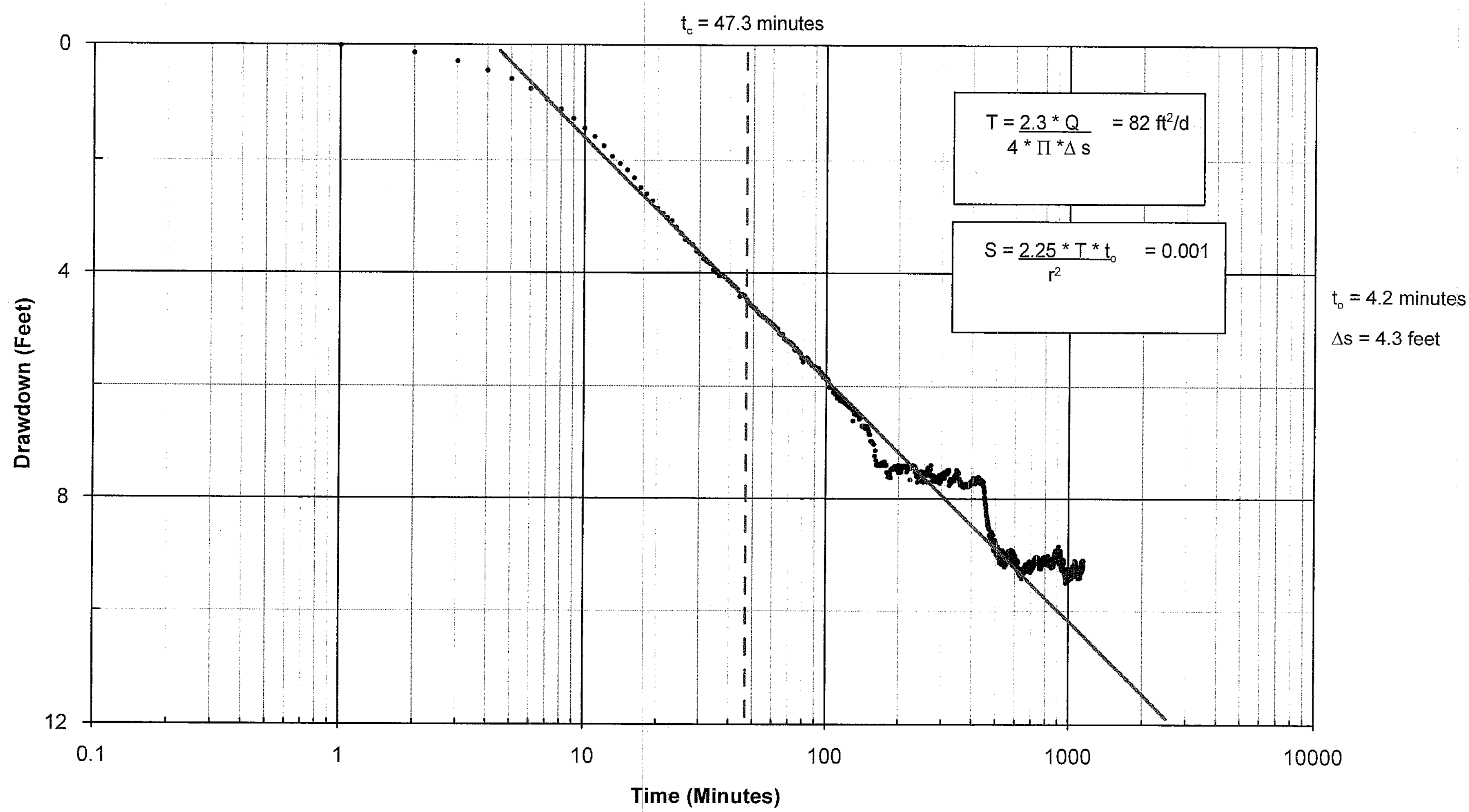
FIG.H-6.2

FIG.H-6.2



NOTE: See Report for discussion of the Cooper Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-1-10 DEEP VWP
COOPER-JACOB ANALYSIS

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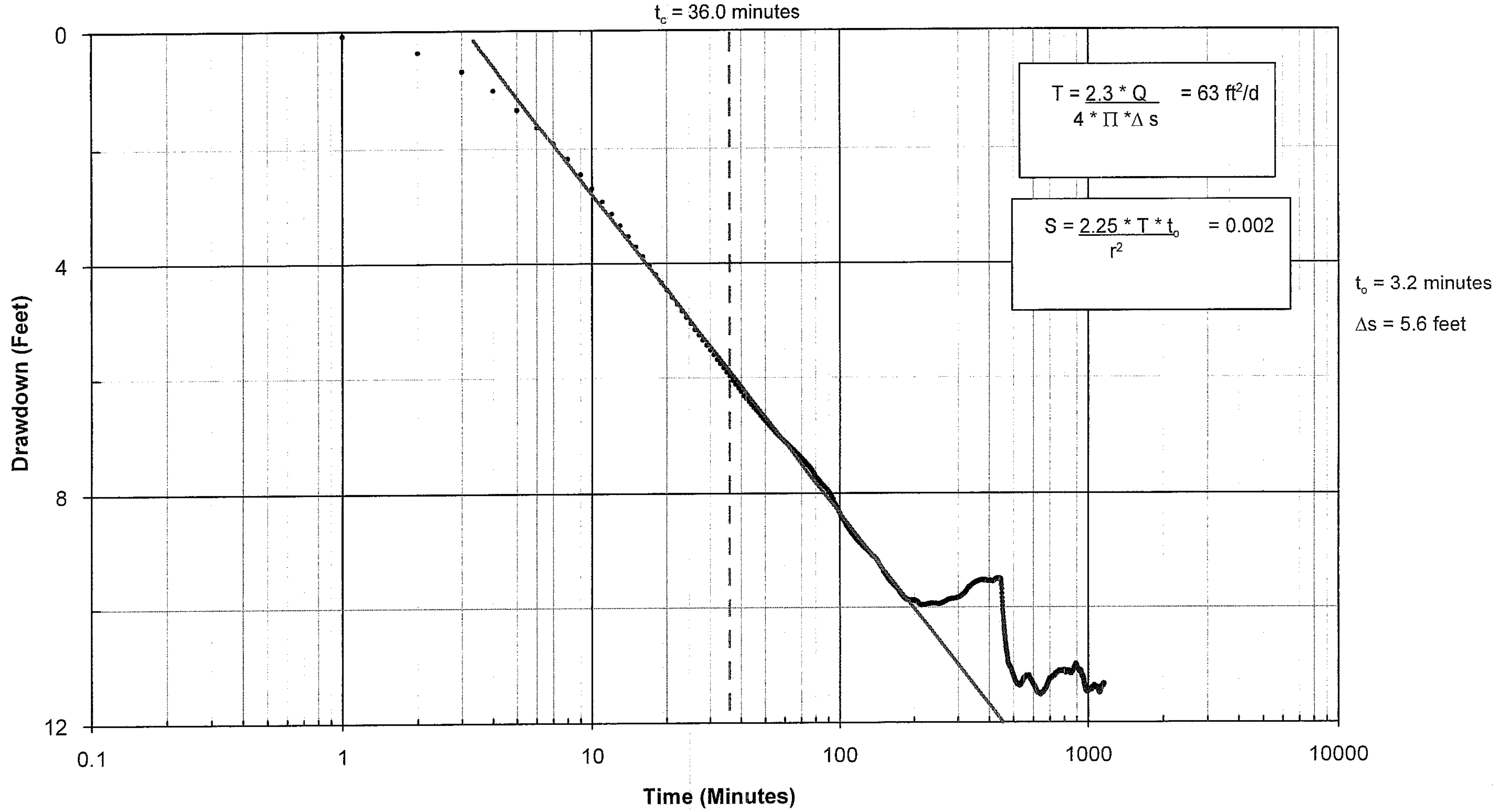
FIG. H-6.3

FIG. H-6.3



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-2-10 DEEP WELL
COOPER-JACOB ANALYSIS

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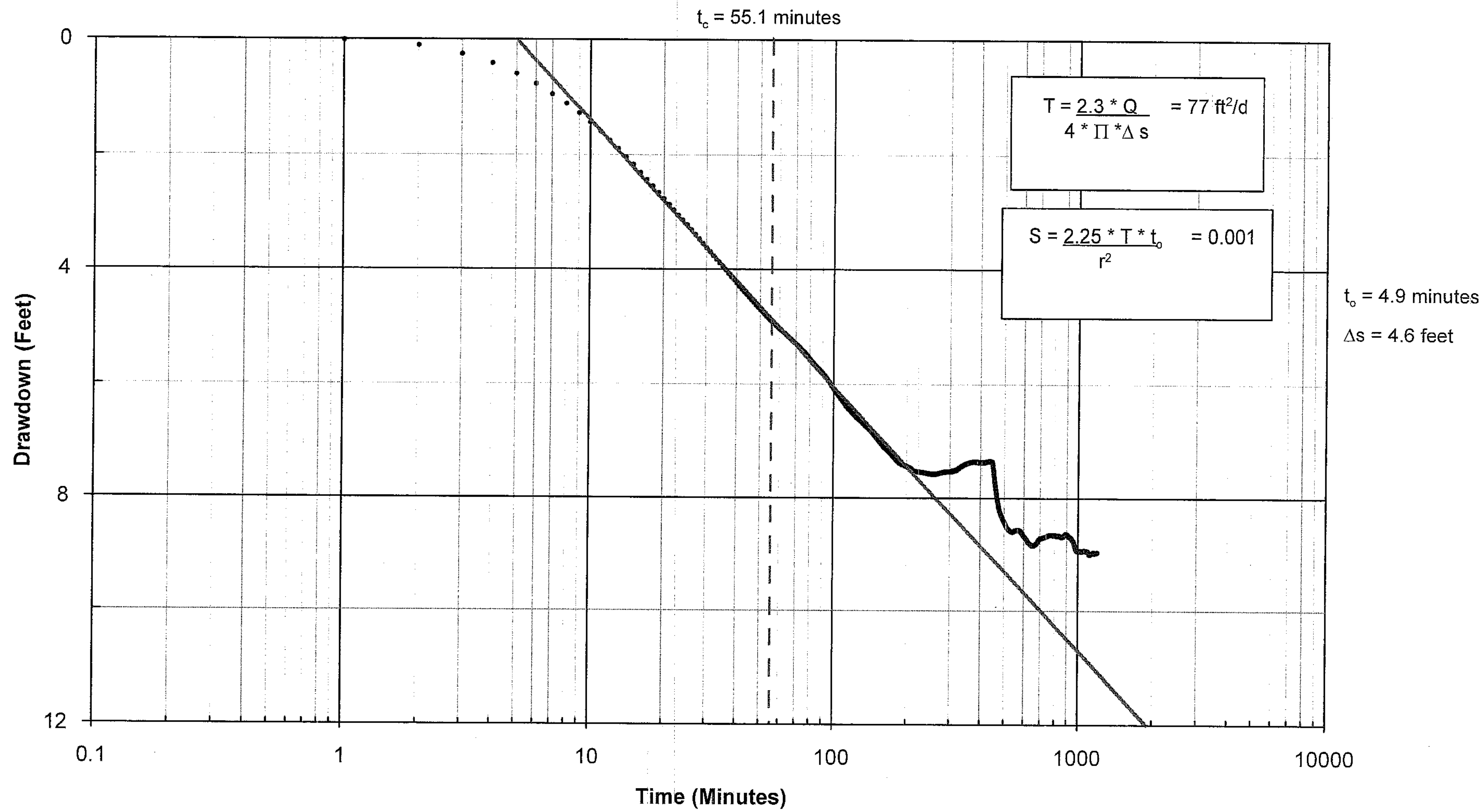
FIG. H-6.4

FIG. H-6.4



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzingpumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-3-10 DEEP WELL
COOPER-JACOB ANALYSIS

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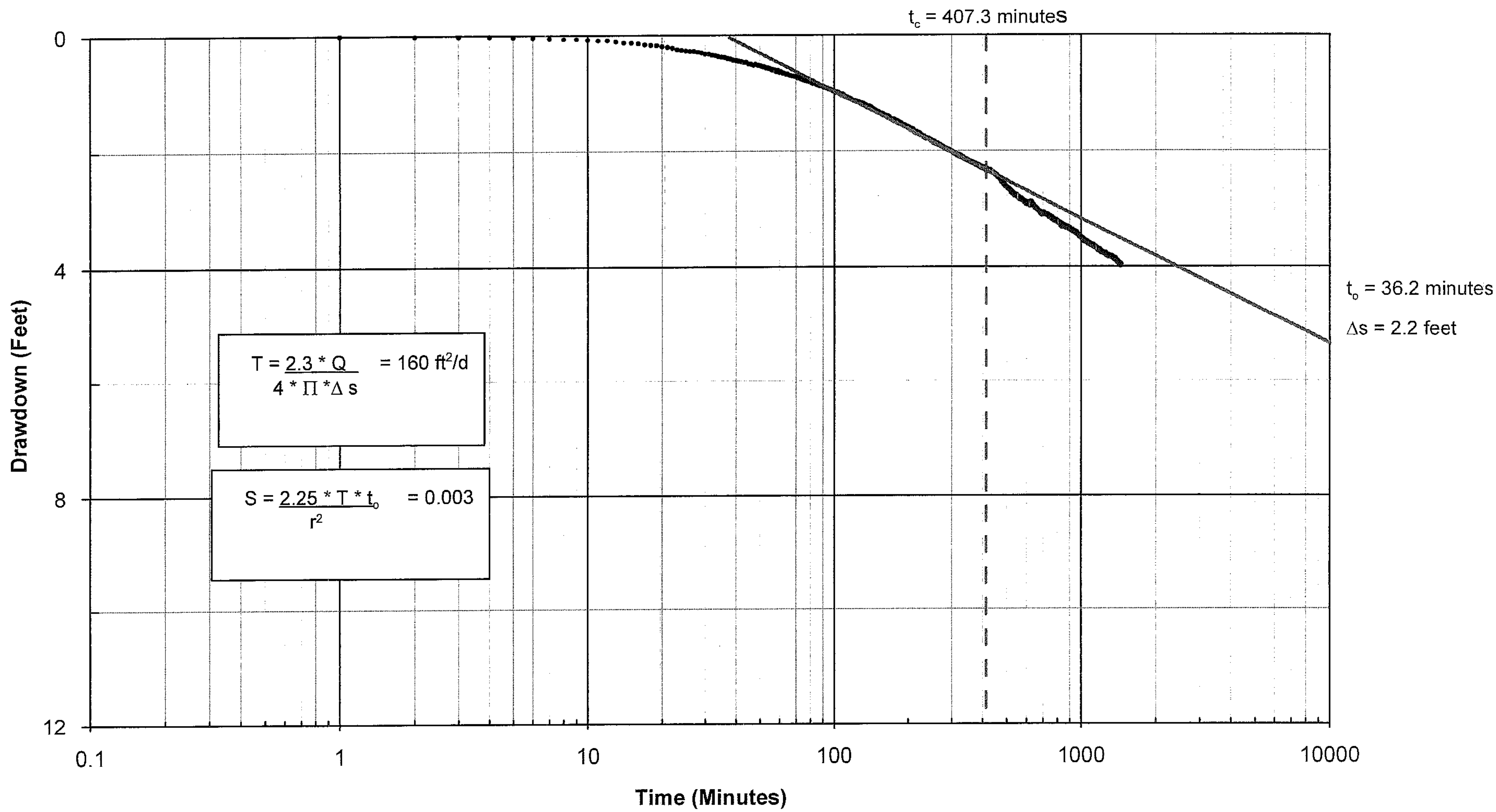
FIG. H-6.5

FIG. H-6.5



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-4-10 DEEP VWP
COOPER-JACOB ANALYSIS

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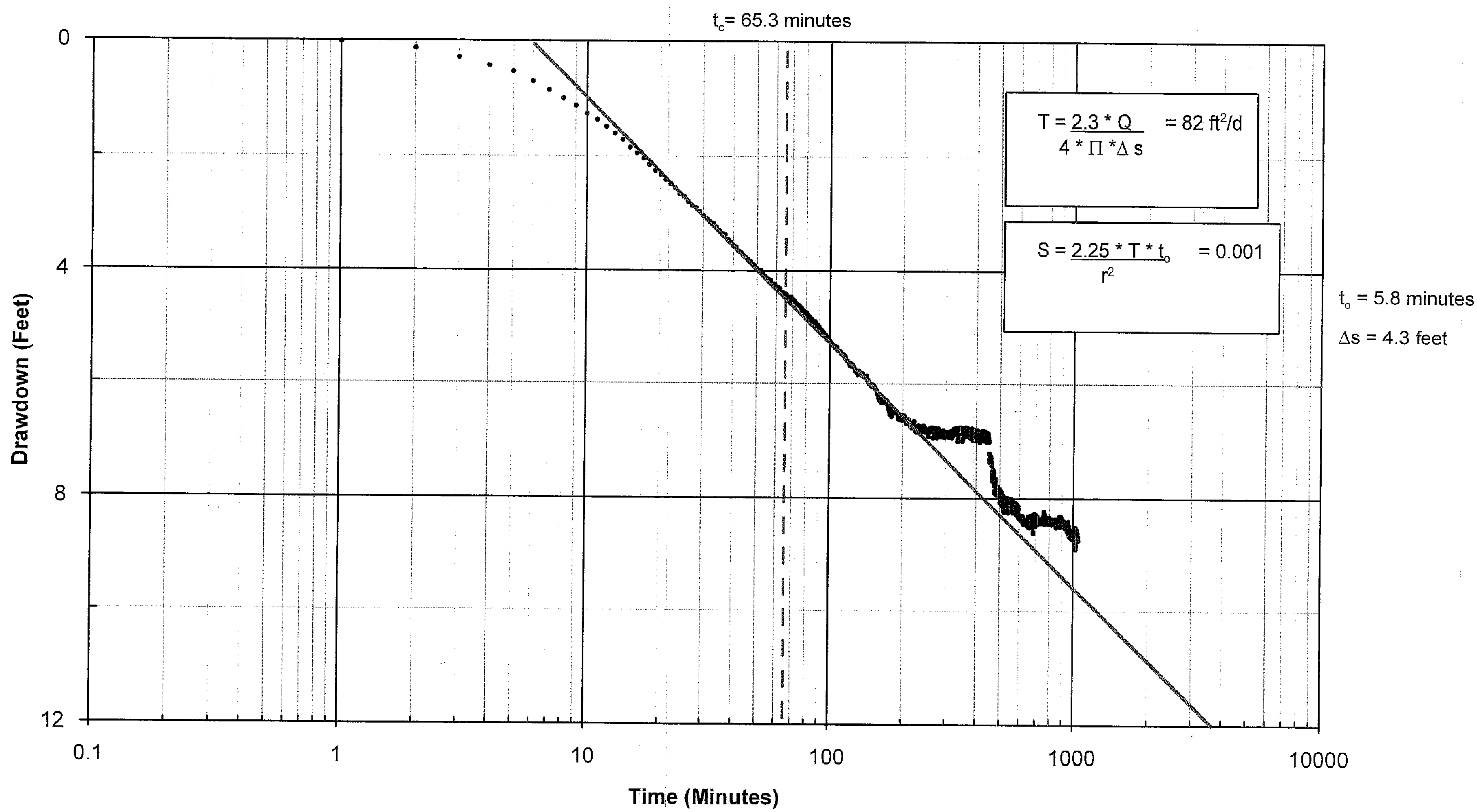
FIG. H-6.6

FIG. H-6.6



NOTE: See Report for disccusion of the Cooper-Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-5-10 DEEP VWP
COOPER-JACOB ANALYSIS

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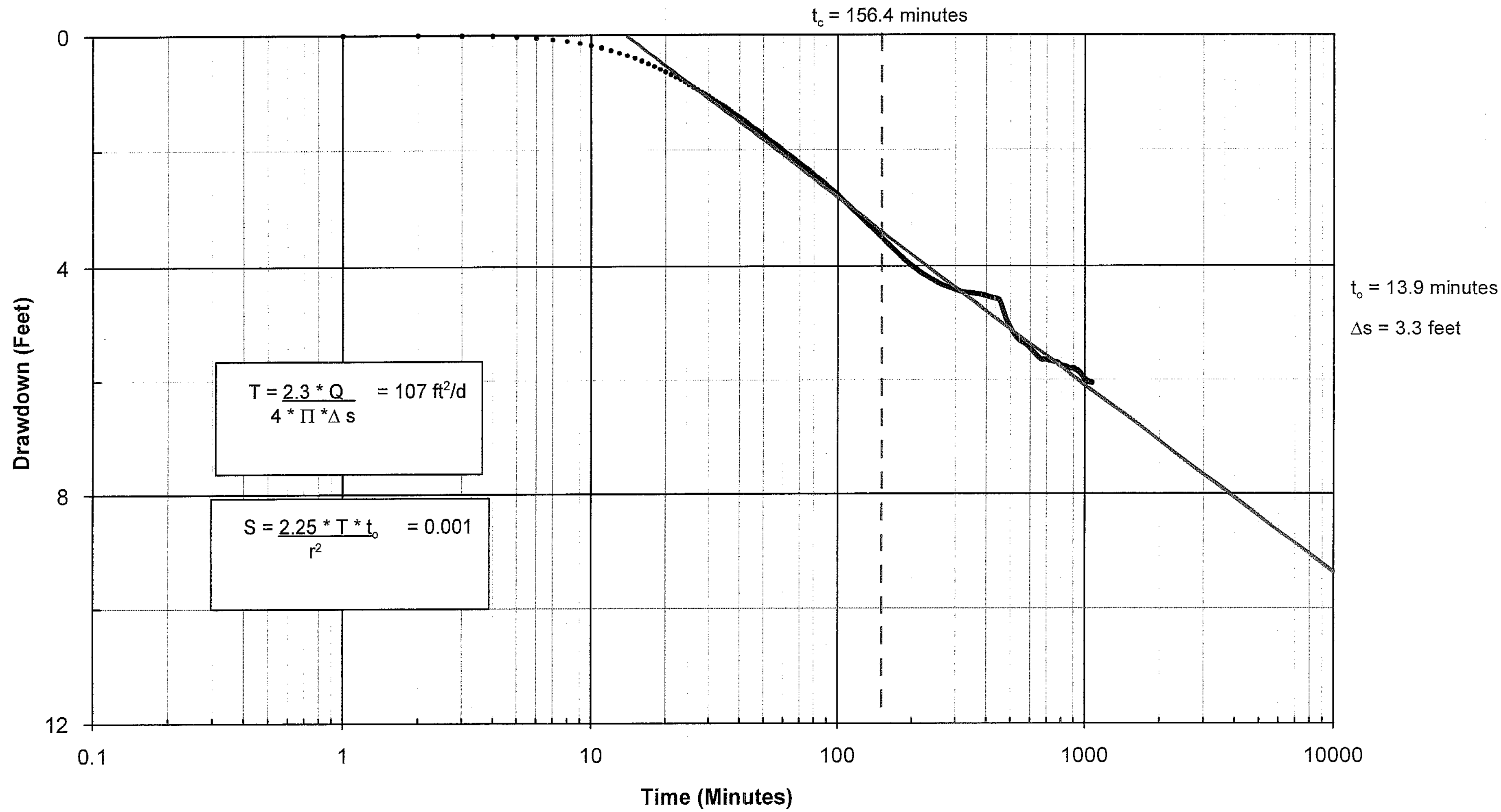
FIG. H-6.7

FIG. H-6.7



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-6-10 DEEP WELL
COOPER-JACOB ANALYSIS

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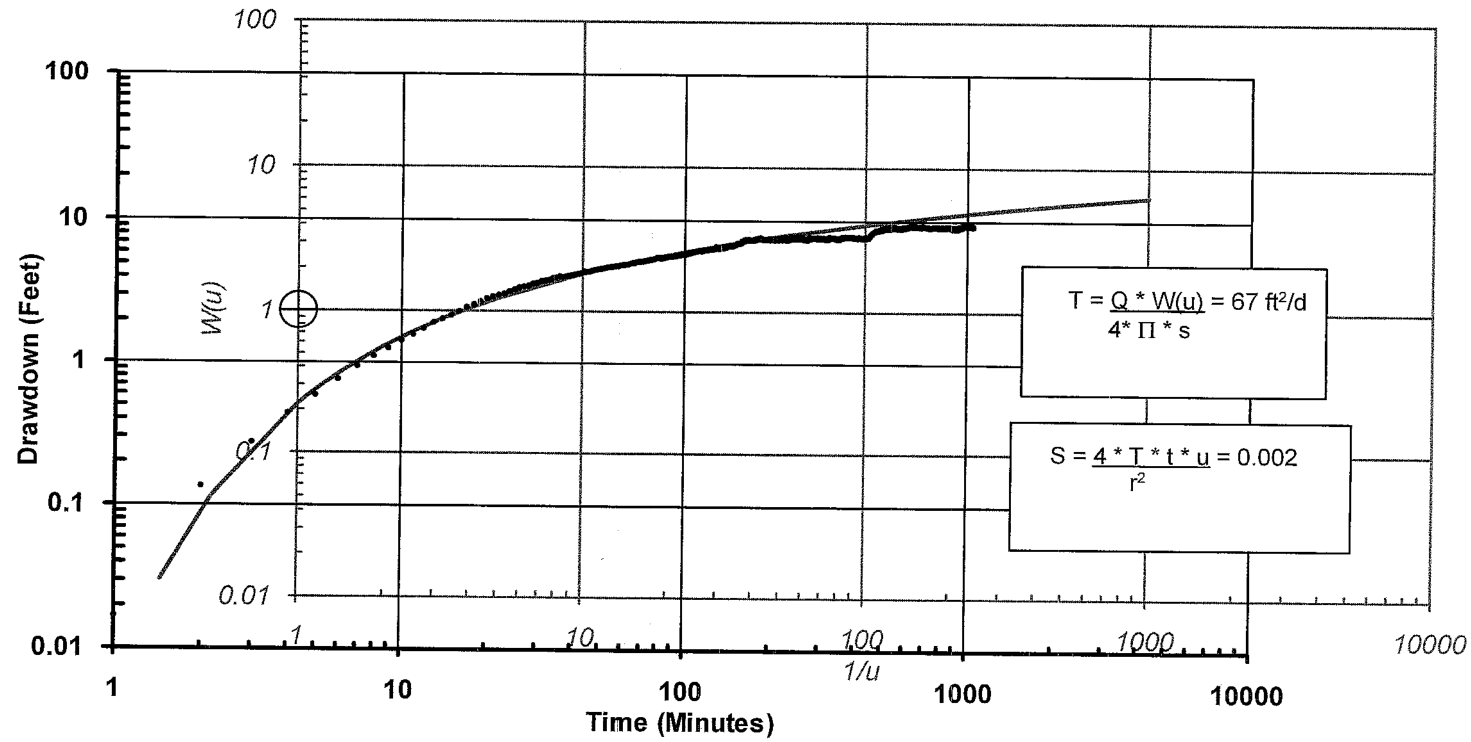
FIG. H-6.8

FIG. H-6.8



- ● Monitoring Well MW-1-10 Deep VWP Drawdown Data
- —— Theis Curve
- ⊕ Match Point

Match Point

W(u) = 1

1/u = 1

t = 4.4 min

s = 2.3 ft

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-1-10 DEEP VWP
THEIS CURVE MATCH

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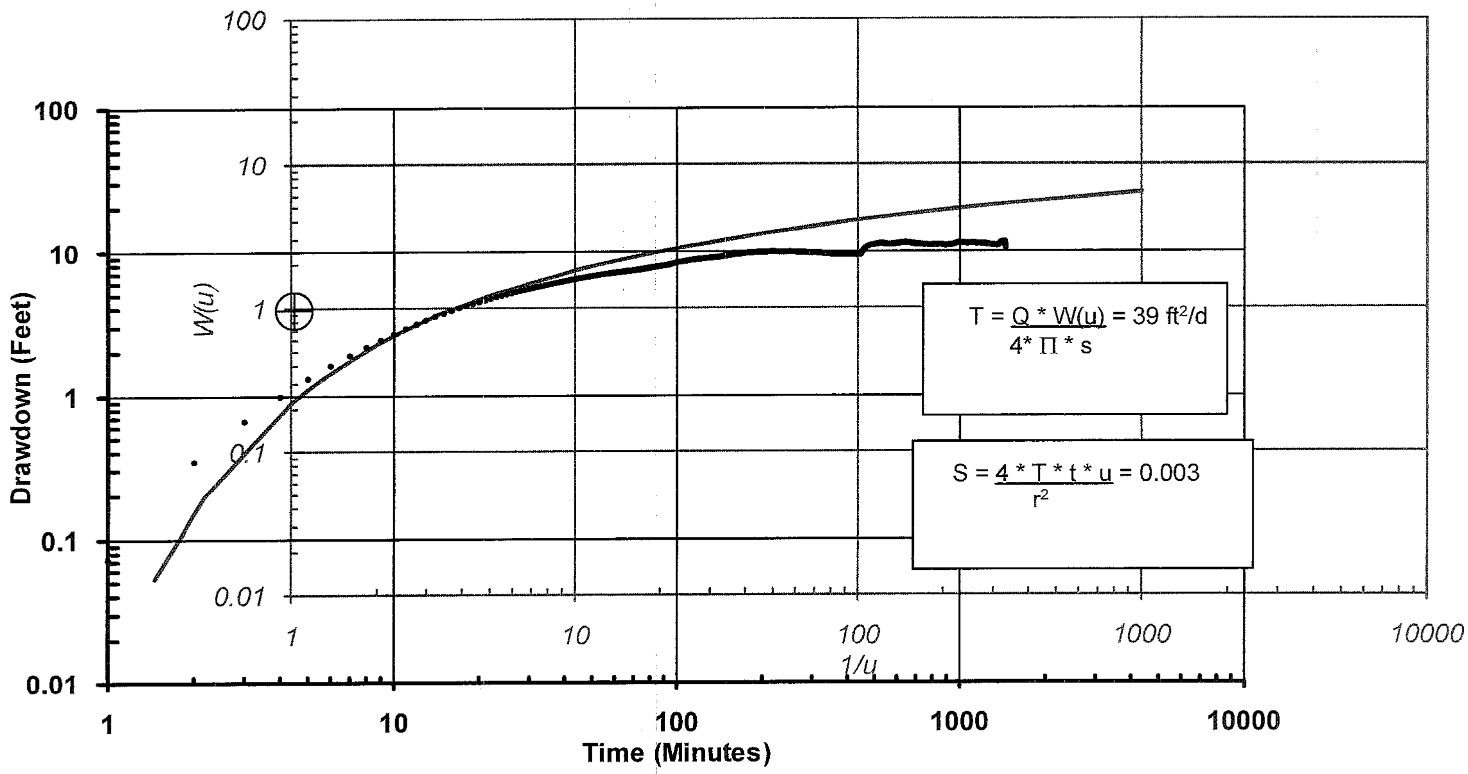
FIG. H-6.9

FIG. H-6.9



- Monitoring Well MW-2-10 Deep Well Drawdown Data
- Theis Curve
- Match Point

Match Point

W(u) = 1

1/u = 1

t = 4.4 min

s = 3.9 ft

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

**PW-4-10 DEEP PUMPING TEST
MW-2-10 DEEP WELL
THEIS CURVE MATCH**

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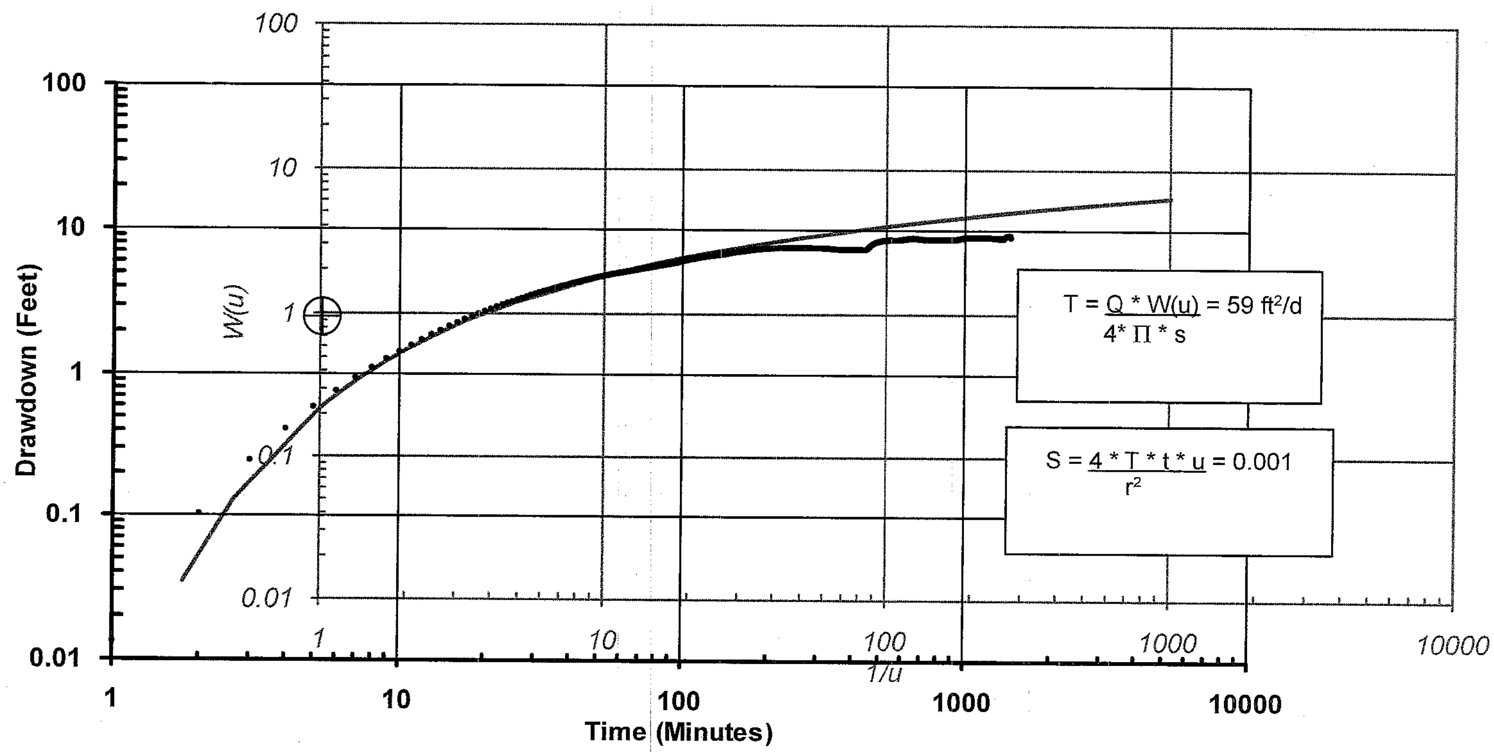
FIG. H-6.10

FIG. H-6.10



● Monitoring Well MW-3-10 Deep Well Drawdown Data

—— Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 5.3 min

s = 2.6 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

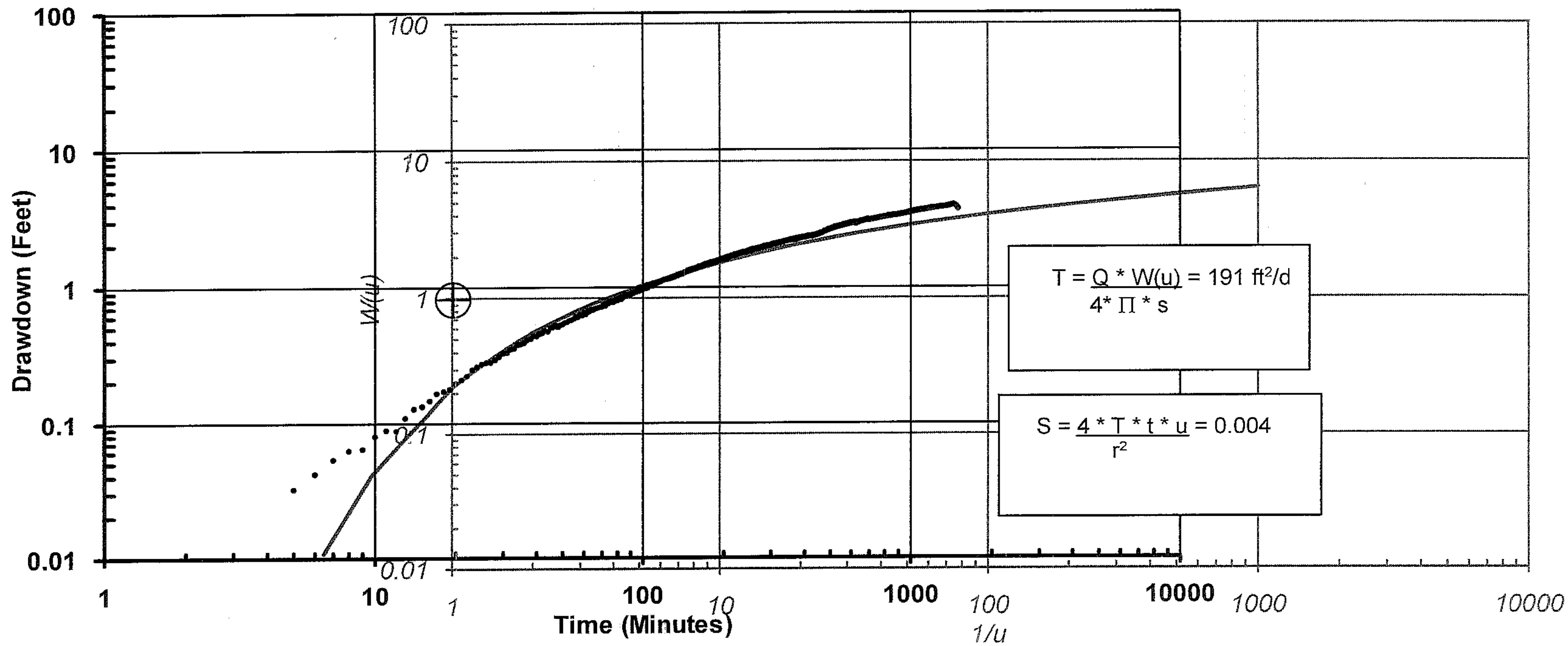
PW-4-10 DEEP PUMPING TEST
MW-3-10 DEEP WELL
THEIS CURVE MATCH

September 2010 | 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-6.11

FIG. H-6.11



● Monitoring Well MW-4-10 Deep VWP Drawdown Data

— Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 19.6 min

s = 0.8 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

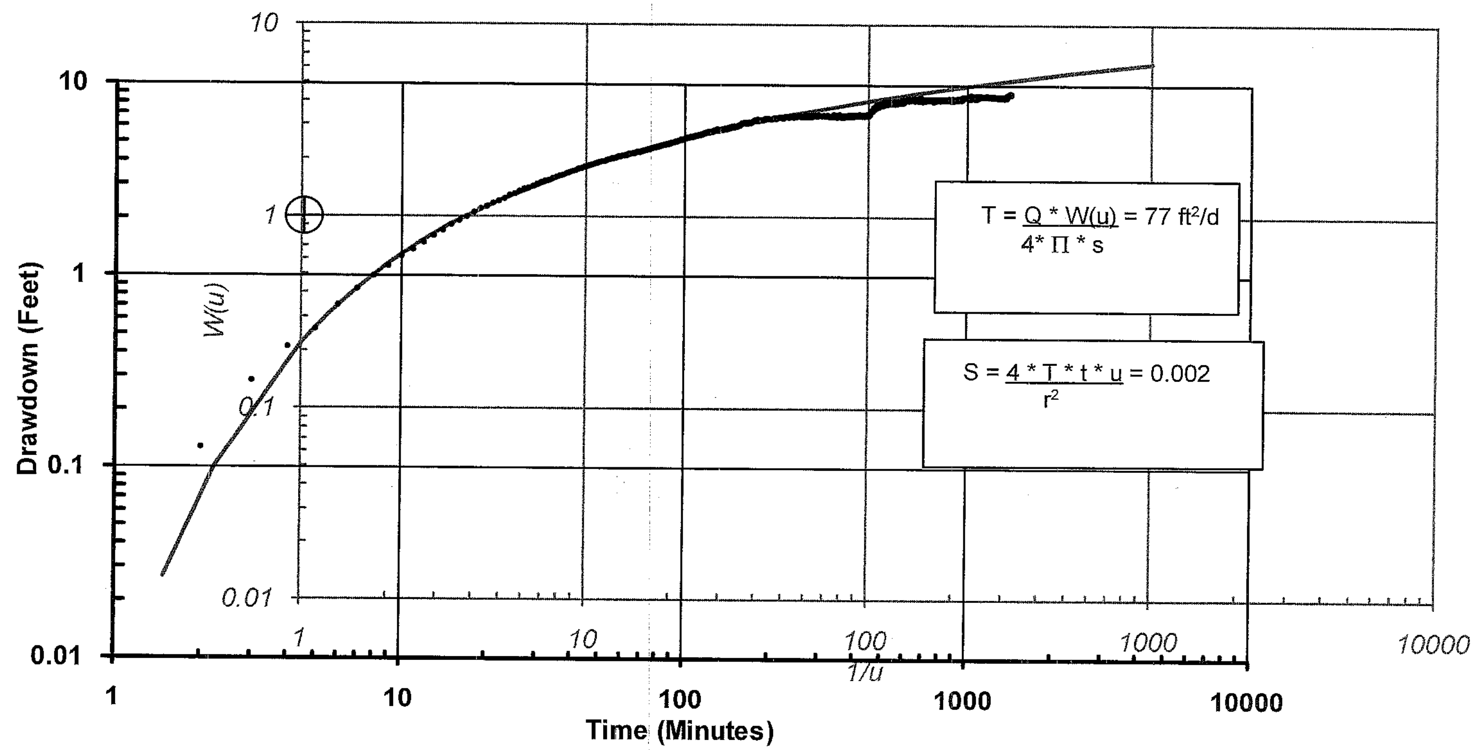
PW-4-10 DEEP PUMPING TEST
MW-4-10 DEEP VWP
THEIS CURVE MATCH

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-6.12

FIG. H-6.12



● Monitoring Well MW-5-10 Deep VWP Drawdown Data

Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analayzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 4.5 min

s = 2.0 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-5-10 DEEP VWP
THEIS CURVE MATCH

September 2010 | 21-1-21190-014

SHANNON & WILSON, INC.
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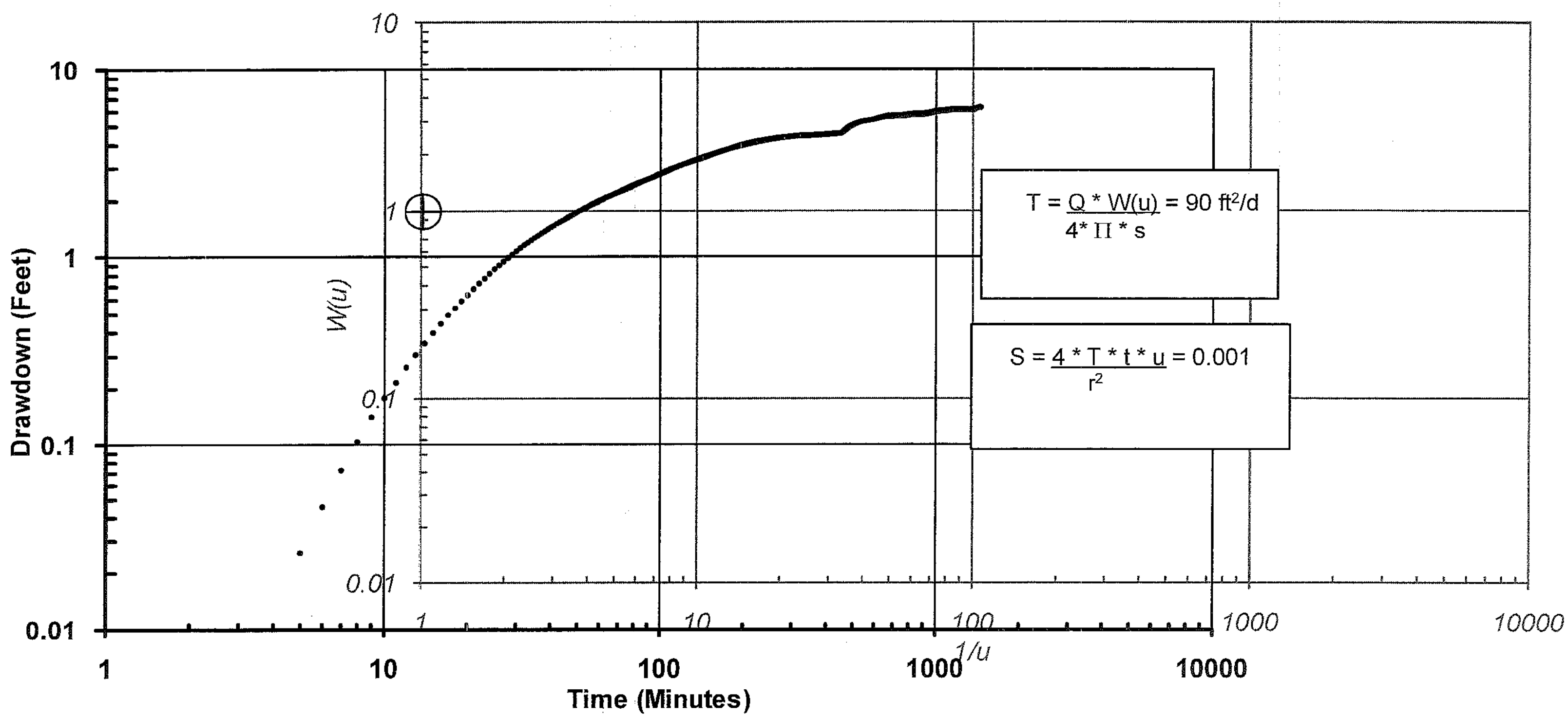
FIG. H-6.13

FIG. H-6.13



● Monitoring Well MW-6-10 Deep Drawdown Data

—— Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analayzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 13.8 min

s = 1.7 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

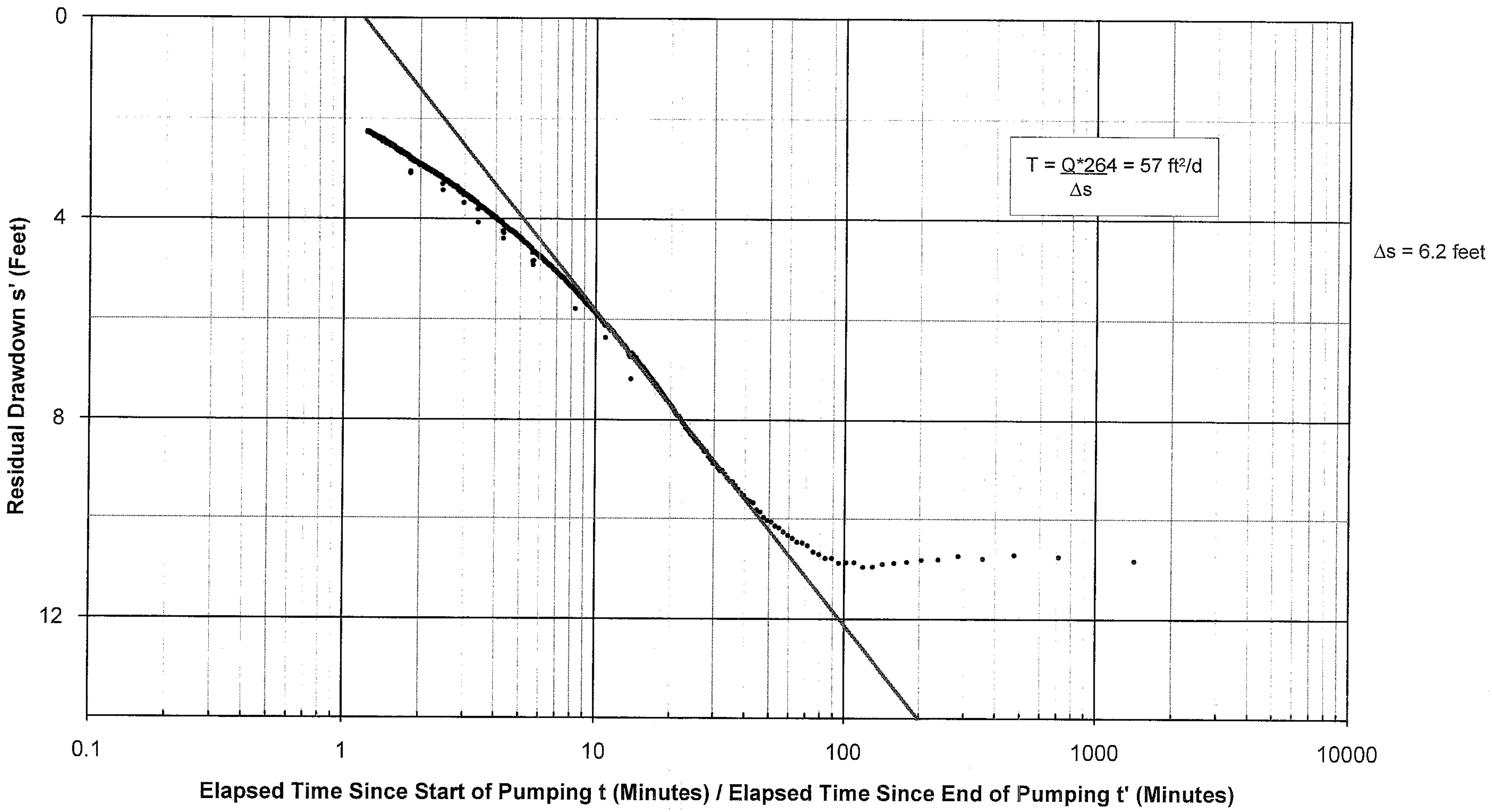
PW-4-10 DEEP PUMPING TEST
MW-6-10 DEEP WELL
THEIS CURVE MATCH

September 2010 | 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-6.14

FIG. H-6.14



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-1-10 DEEP VWP
RECOVERY DATA

September 2010 | 21-1-21190-014

SHANNON & WILSON, INC.
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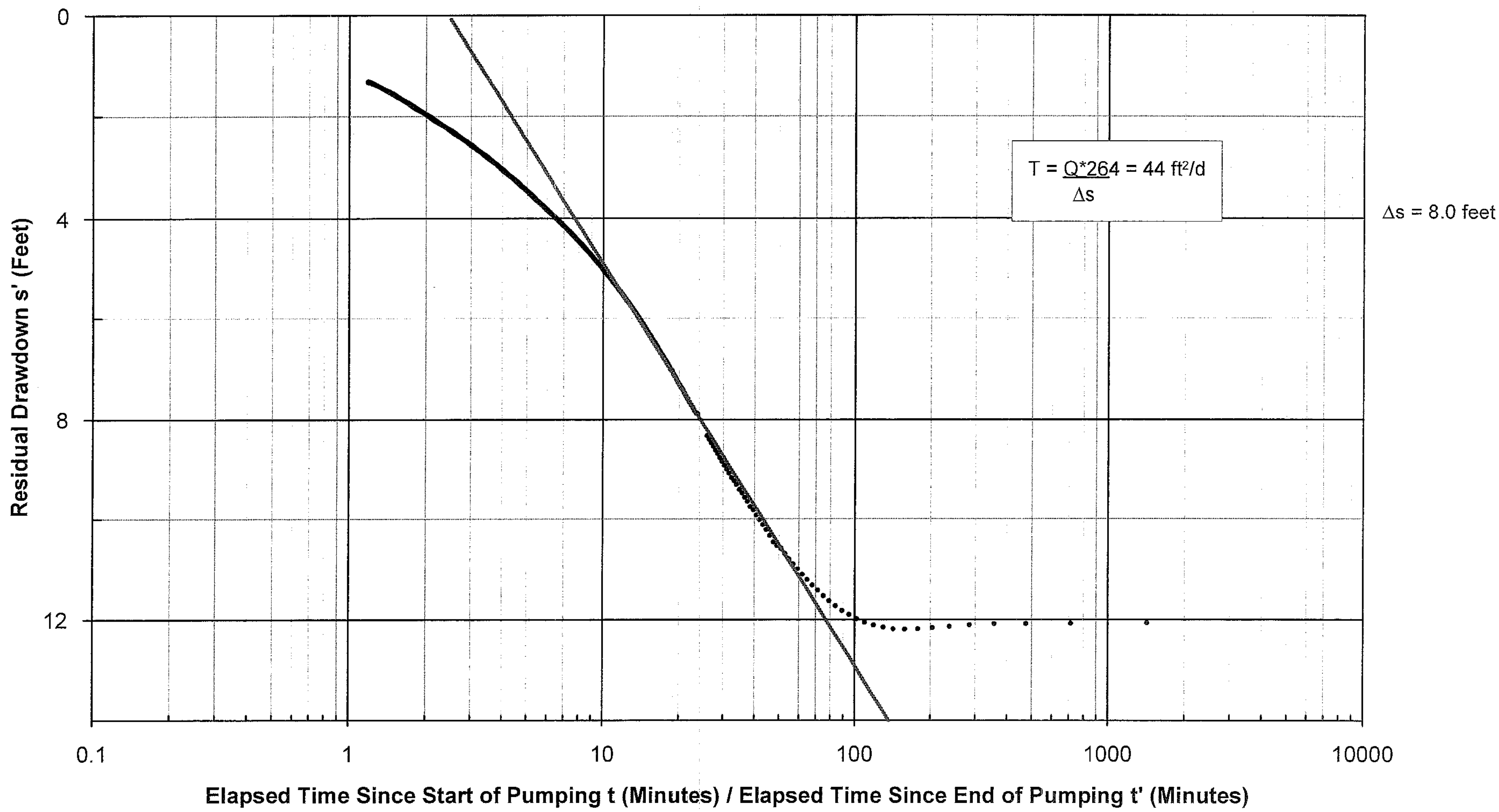
FIG. H-6.15

FIG. H-6.15



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-2-10 DEEP WELL
RECOVERY DATA

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SHANNON & WILSON, INC.
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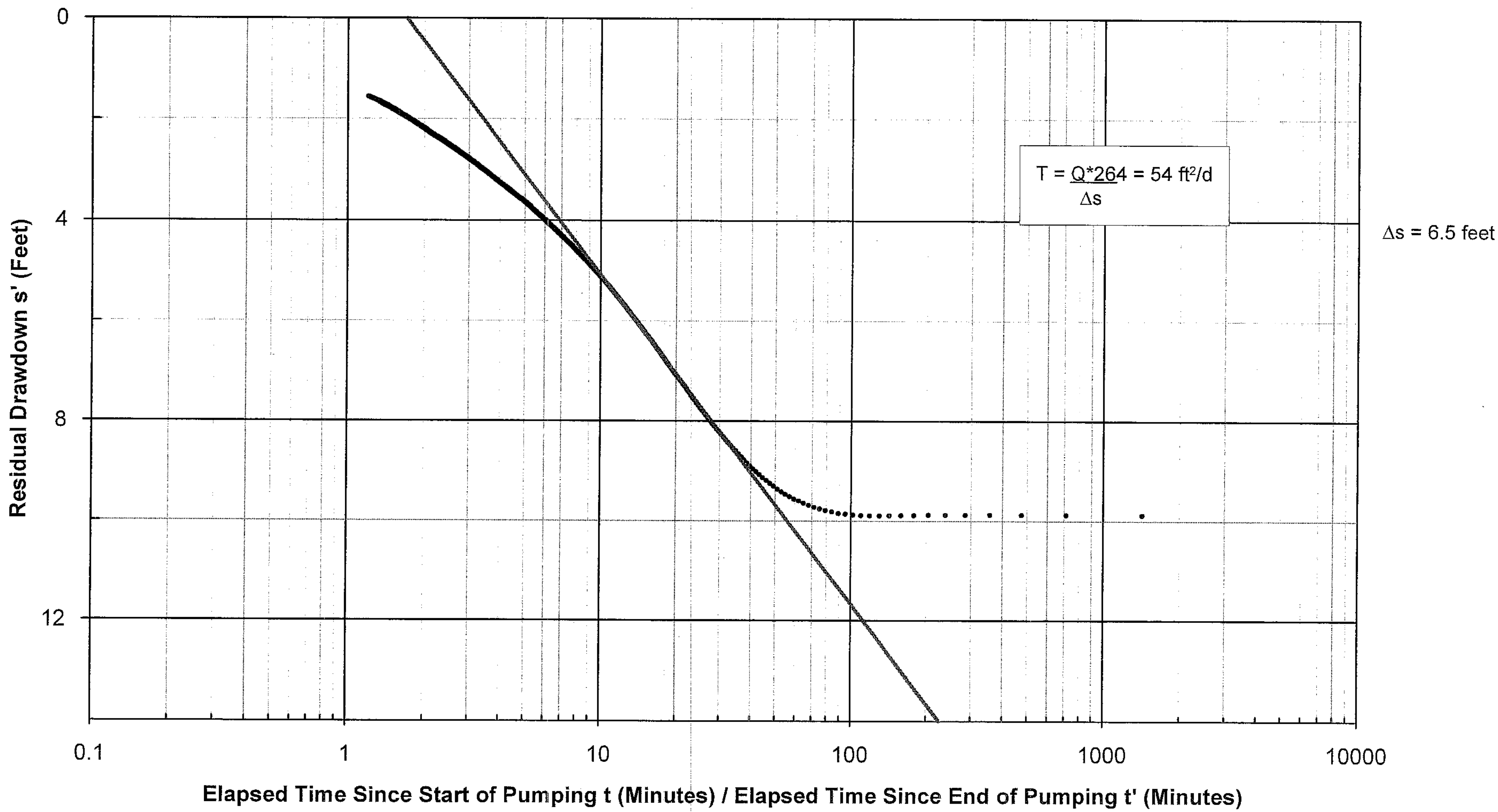
FIG. H-6.16

FIG. H-6.16



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-3-10 DEEP WELL
RECOVERY DATA

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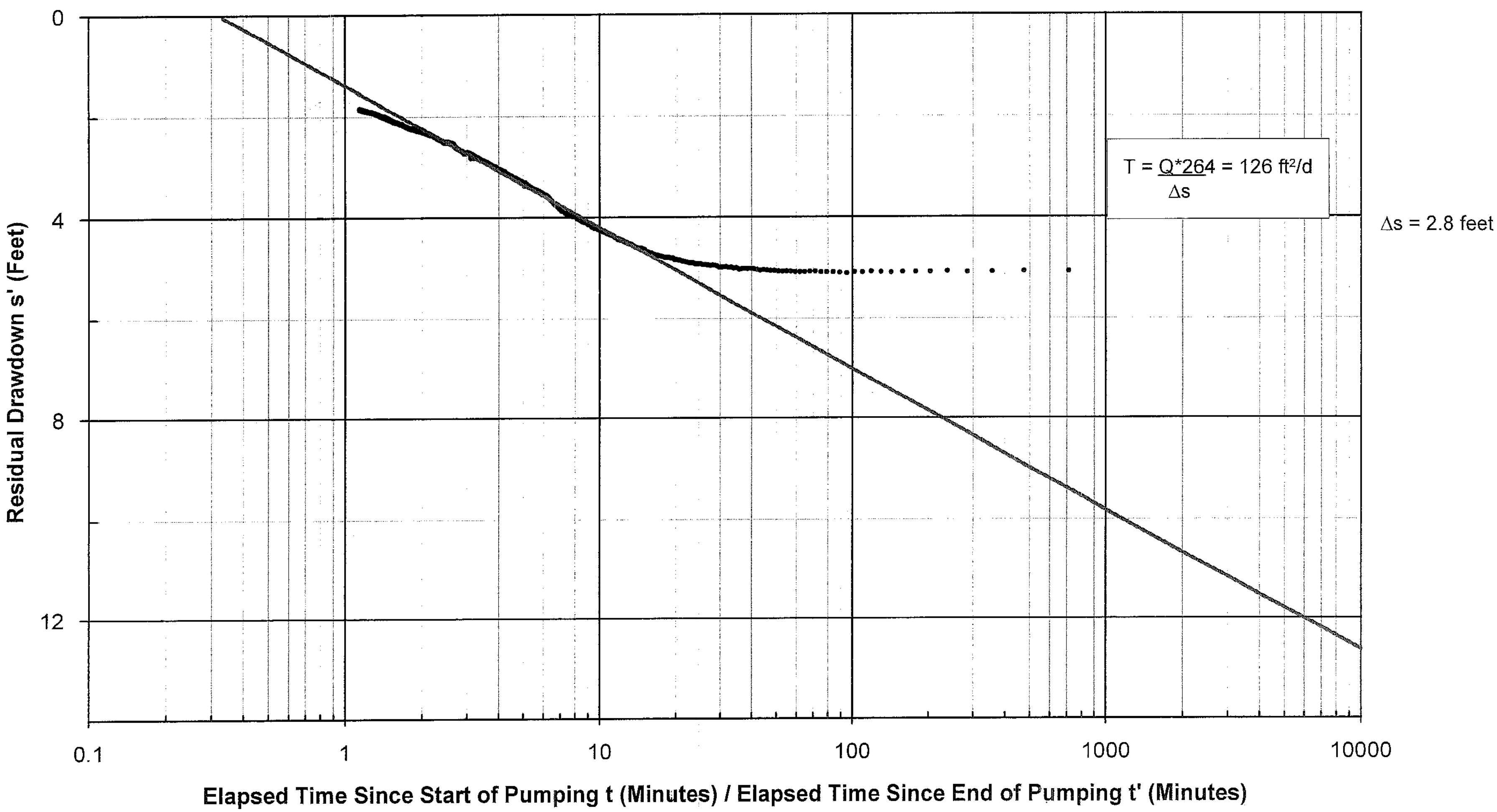
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-6.17

FIG. H-6.17



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-4-10 DEEP VWP
RECOVERY DATA

September 2010 | 21-1-21190-014

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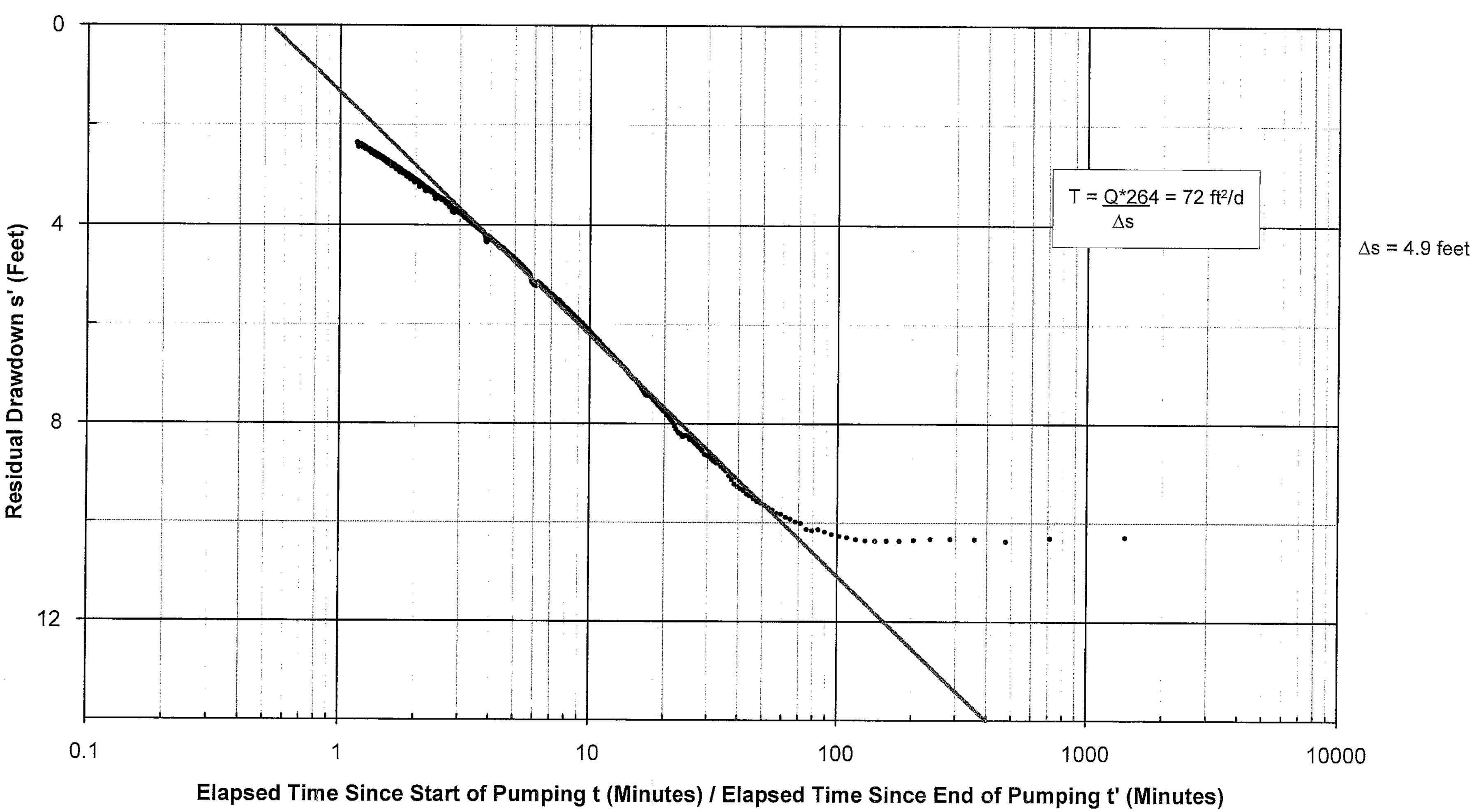
FIG. H-6.18

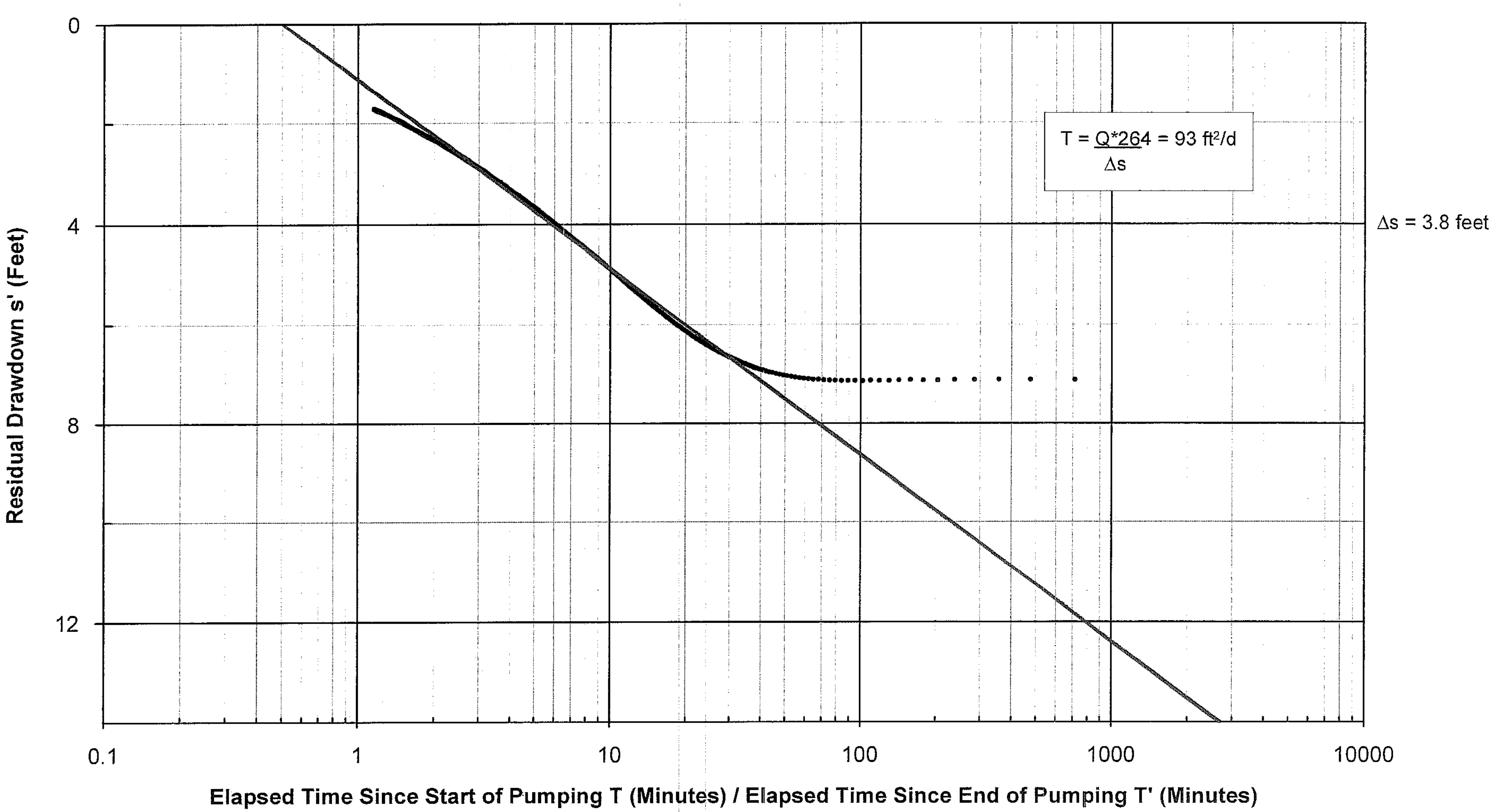
FIG. H-6.18



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility Aberdeen, Washington	
PW-4-10 DEEP PUMPING TEST **MW-5-10 DEEP VWP** **RECOVERY DATA**	
September 2010	21-1-21190-014
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. H-6.19**

FIG. H-6.19



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-4-10 DEEP PUMPING TEST
MW-6-10 DEEP WELL
RECOVERY DATA

September 2010 | 21-1-21190-014

SHANNON & WILSON, INC.
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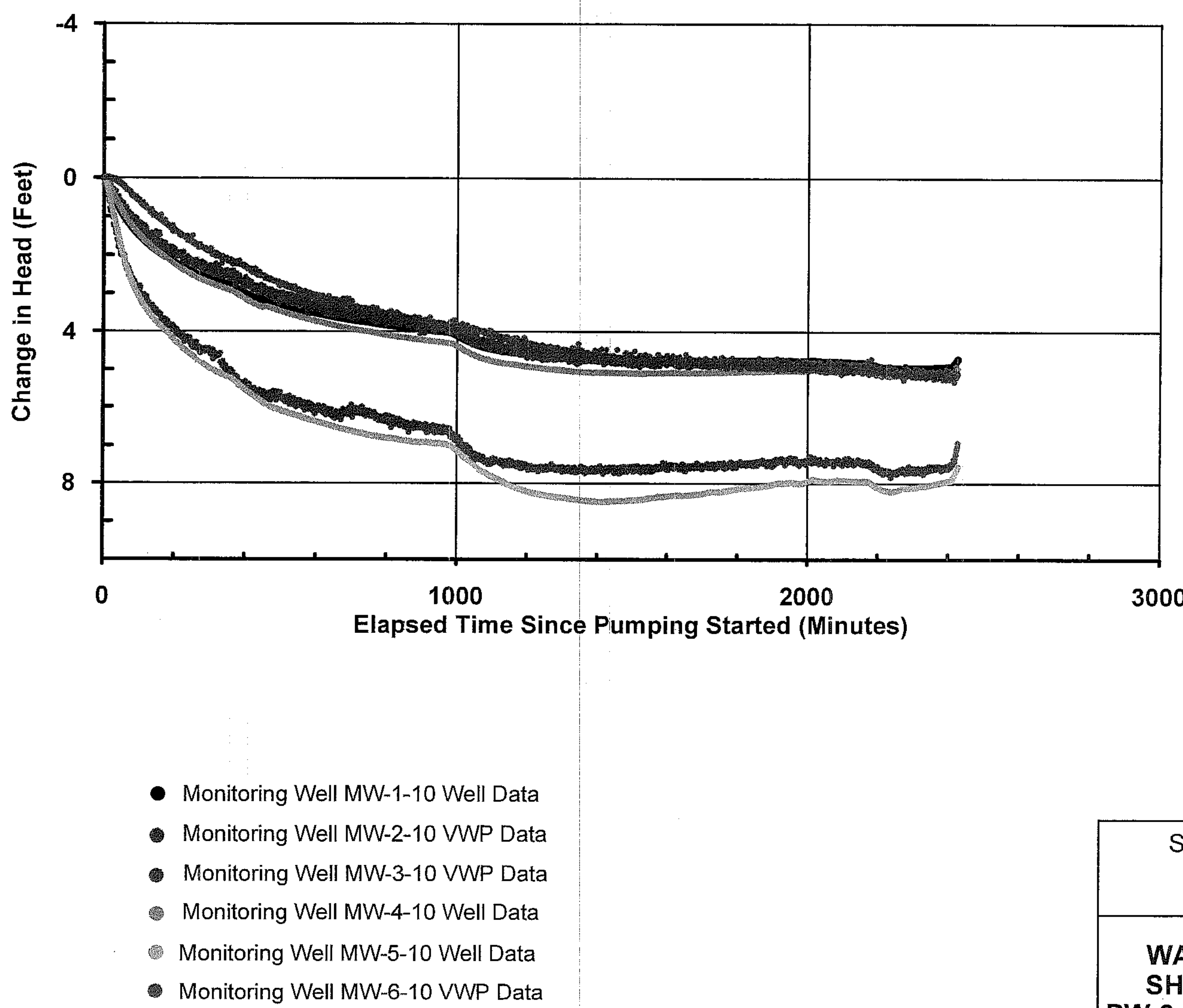
FIG. H-6.20

FIG. H-6.20



SR 520 Pontoon Casting Facility
Aberdeen, Washington

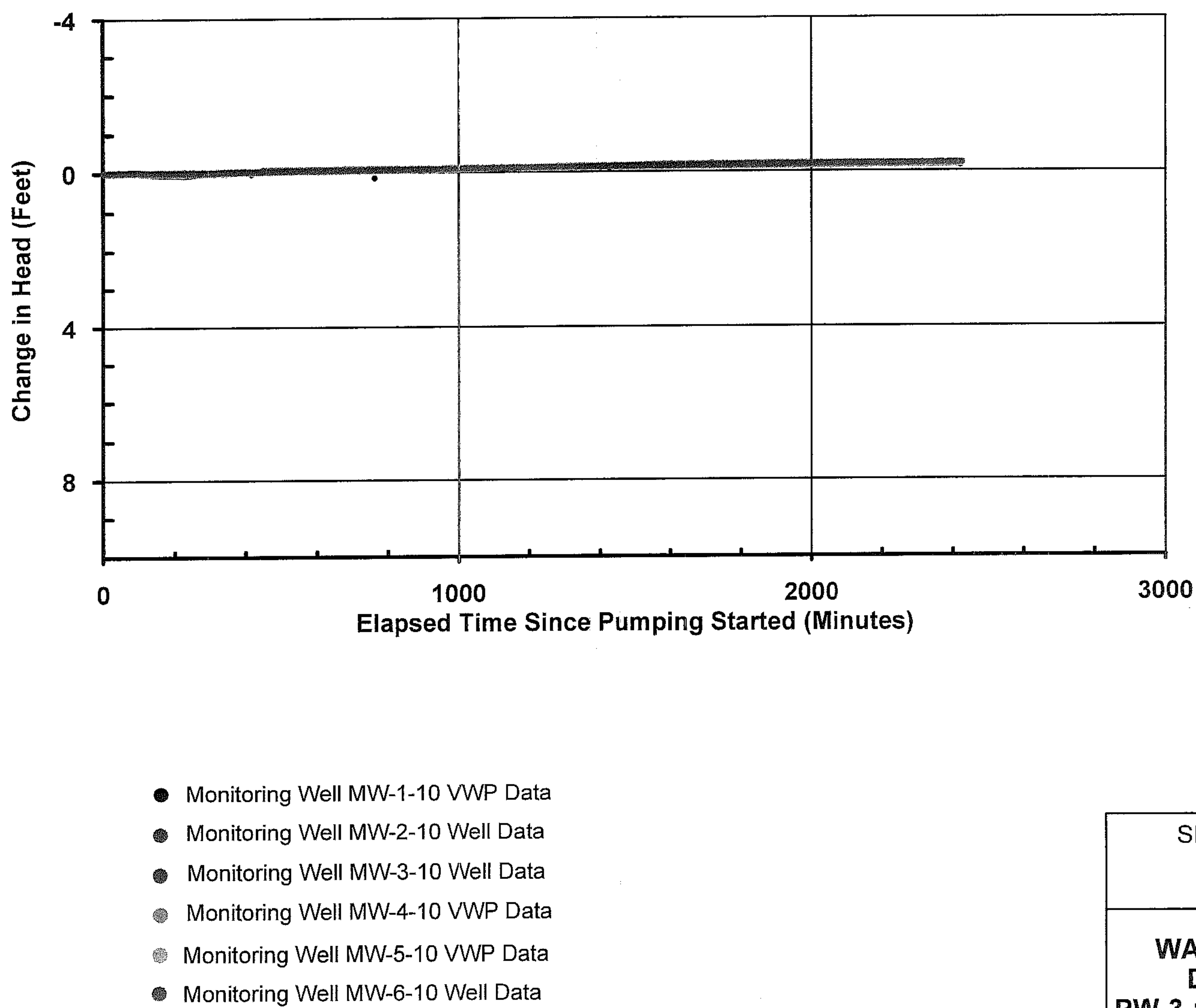
WATER LEVEL HYDROGRAPH SHALLOW INSTRUMENTATION PW-3-10 SHALLOW PUMPING TEST 2

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG.H-7.1

FIG.H-7.1



SR 520 Pontoon Casting Facility
Aberdeen, Washington

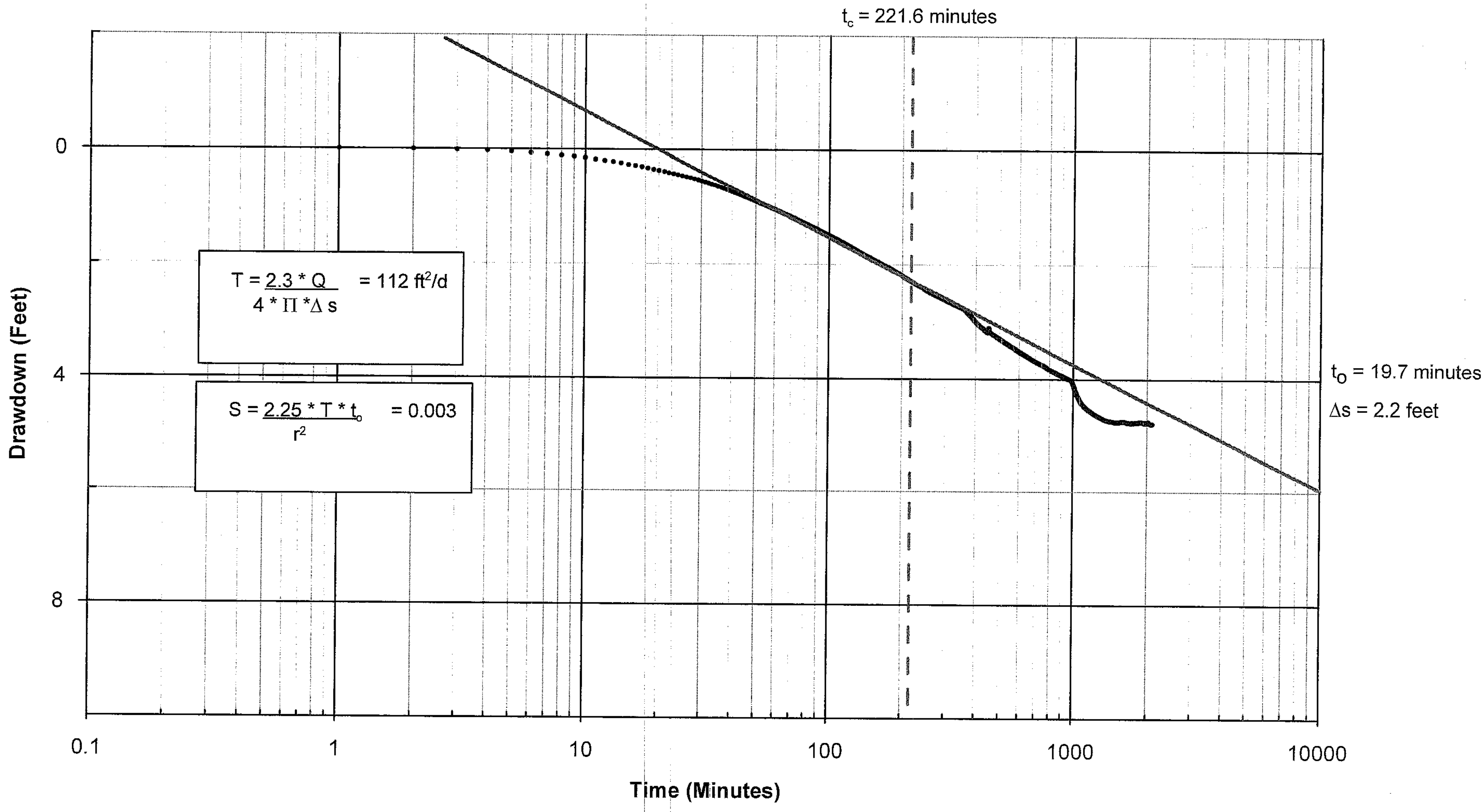
WATER LEVEL HYDROGRAPH
DEEP INSTRUMENTATION
PW-3-10 SHALLOW PUMPING TEST 2

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG.H-7.2

FIG.H-7.2



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 2
MW-1-10 SHALLOW WELL
COOPER-JACOB ANALYSIS

September 2010 | 21-1-21190-014

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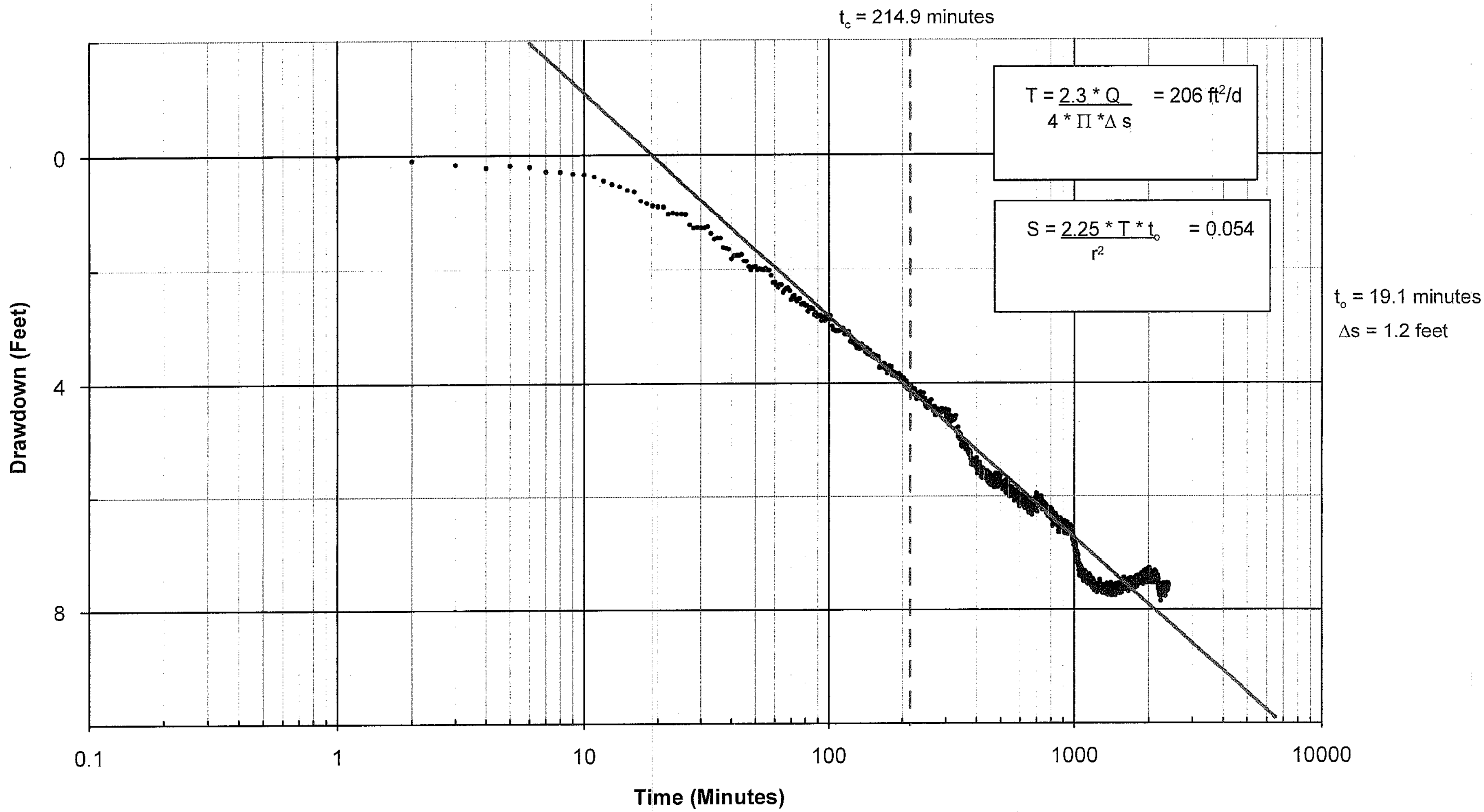
FIG. H-7.3

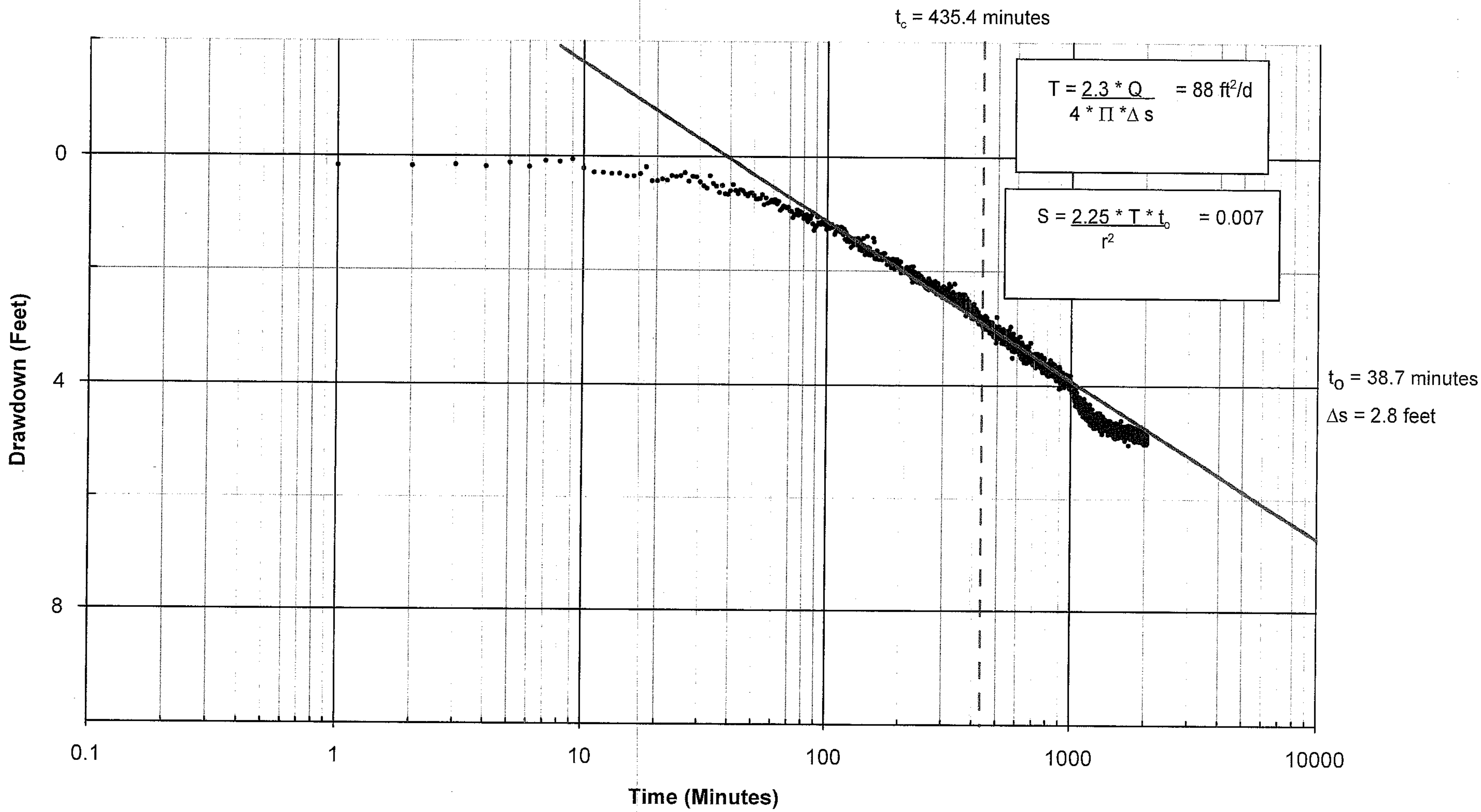
FIG. H-7.3



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility Aberdeen, Washington
PW-3-10 SHALLOW PUMPING TEST 2 MW-2-10 SHALLOW VWP COOPER-JACOB ANALYSIS
September 2010 — 21-1-21190-014
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants — **FIG. H-7.4**

FIG. H-7.4



NOTE: See Report for discussion of Cooper-Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

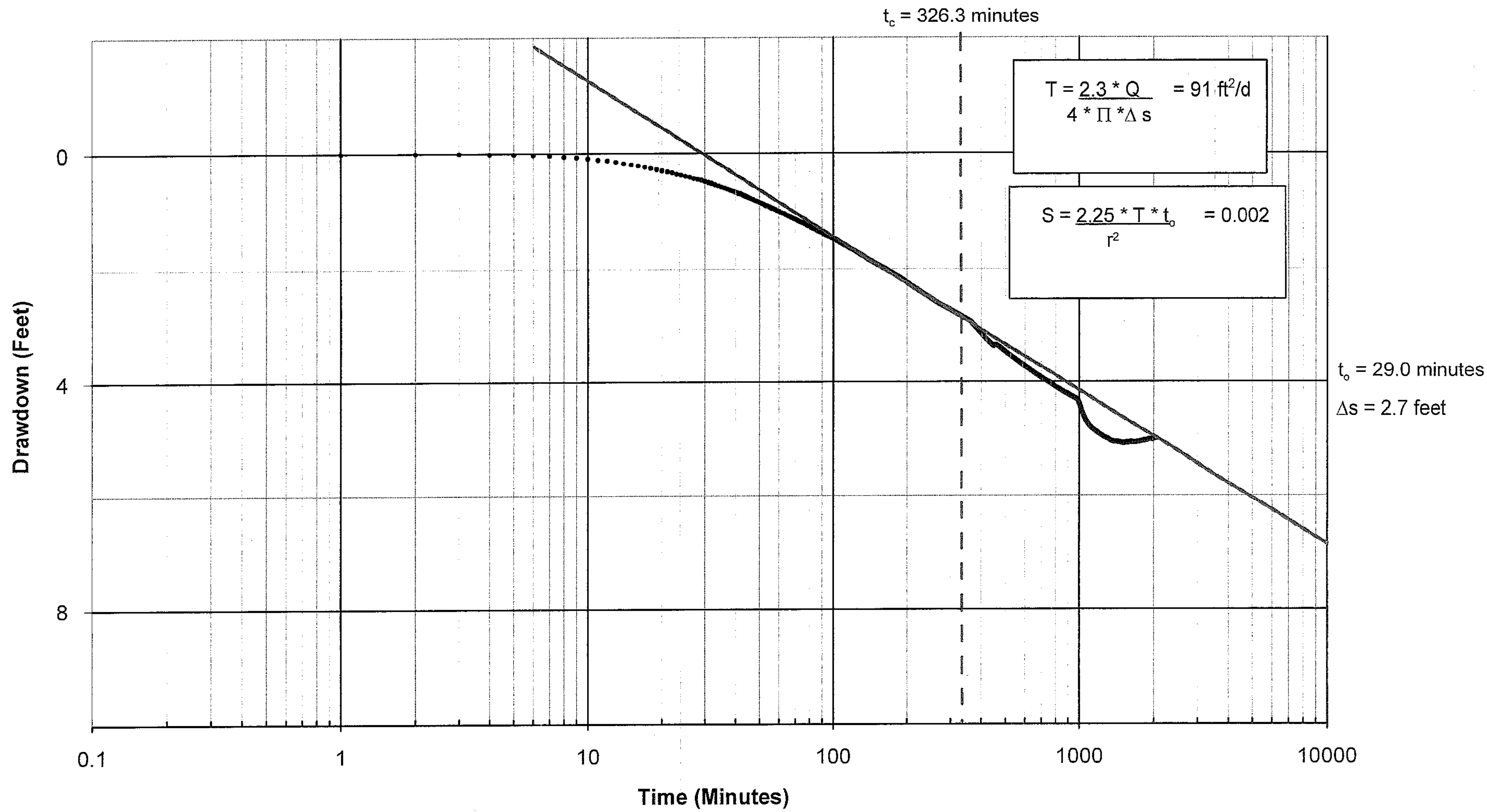
PW-3-10 SHALLOW PUMPING TEST 2
MW-3-10 SHALLOW VWP
COOPER-JACOB ANALYSIS

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-7.5

FIG. H-7.5



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 2
MW-4-10 SHALLOW WELL
COOPER-JACOB ANALYSIS

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
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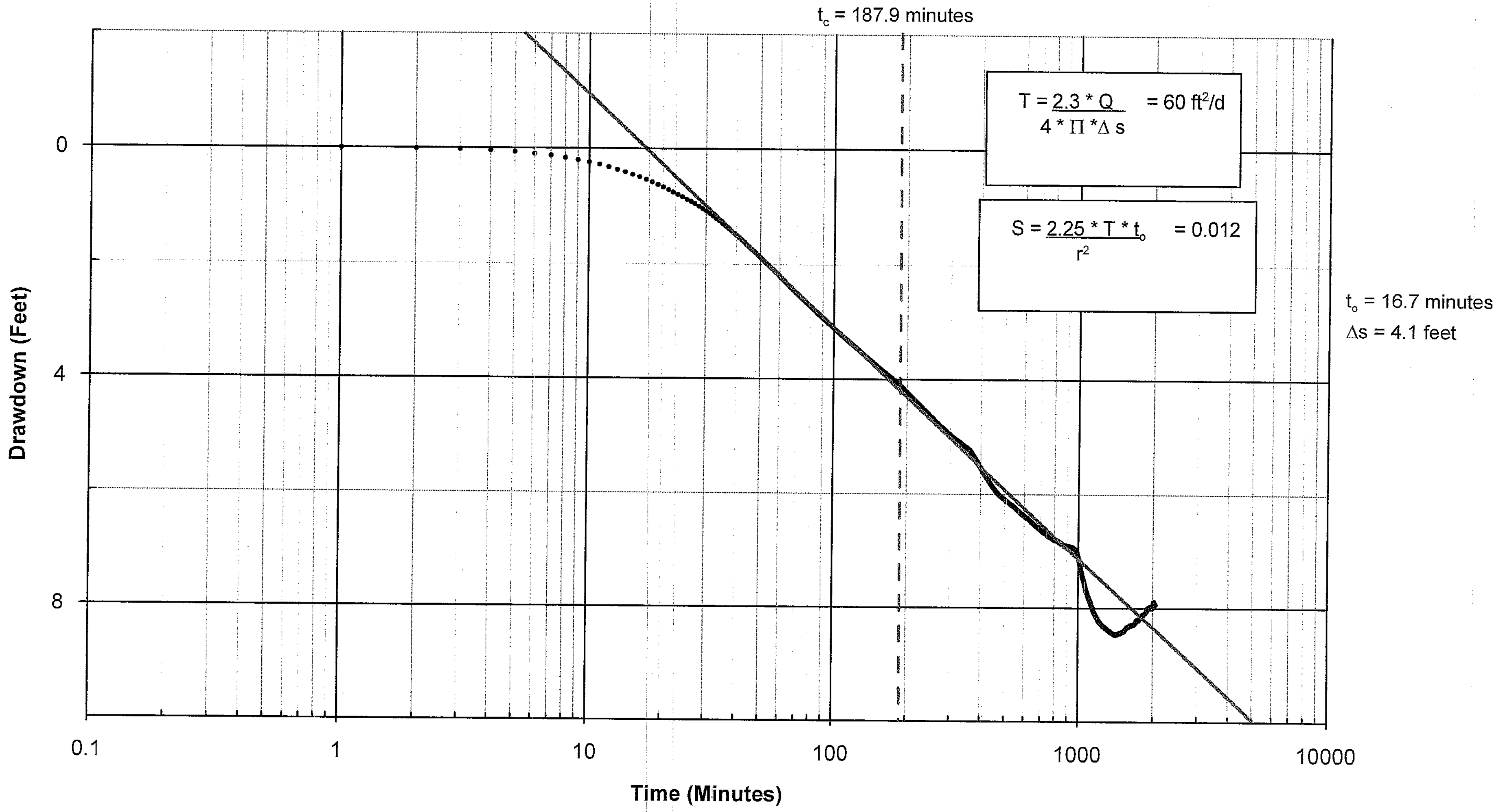
FIG. H-7.6

FIG. H-7.6



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

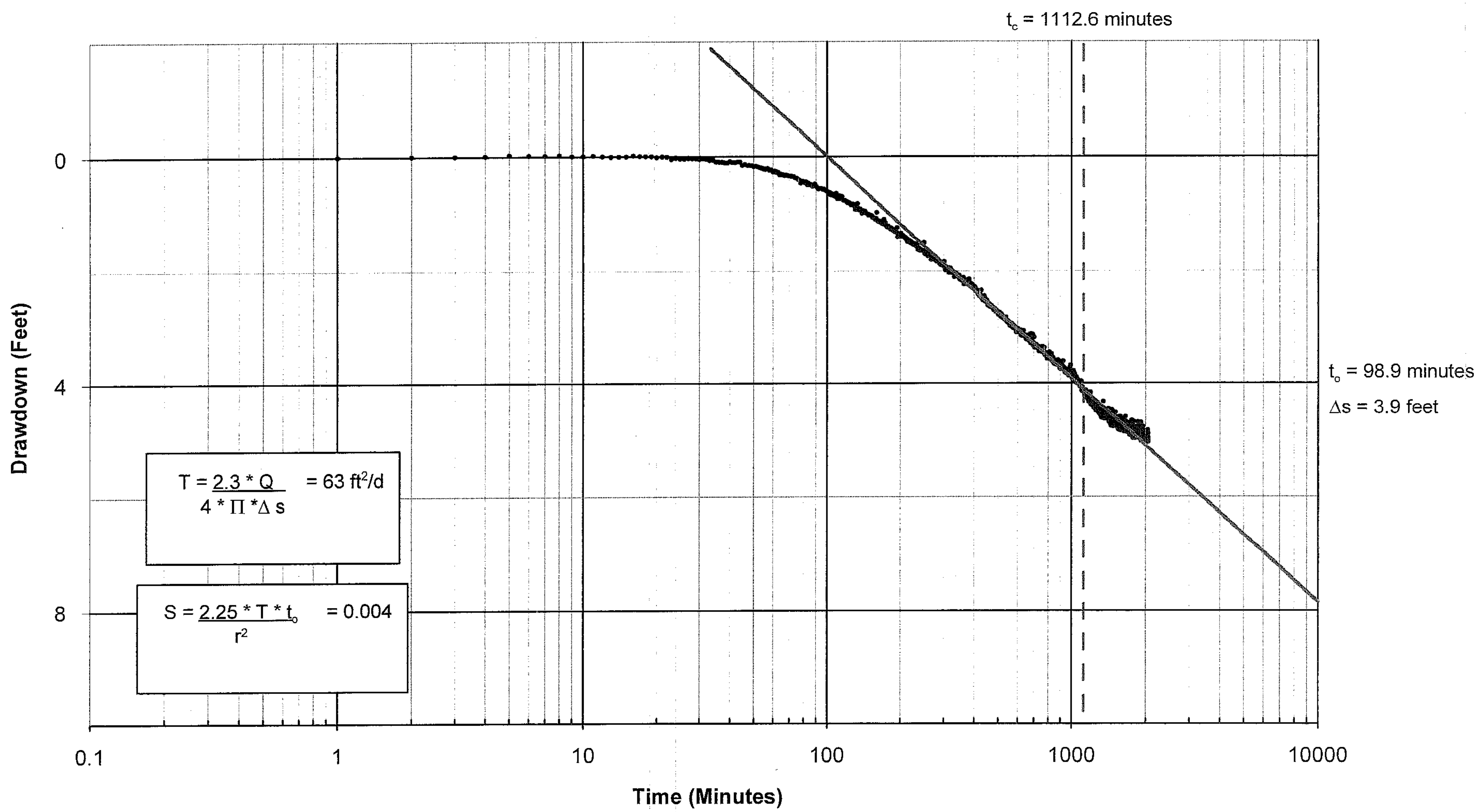
PW-3-10 SHALLOW PUMPING TEST 2
MW-5-10 SHALLOW WELL
COOPER-JACOB ANALYSIS

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-7.7

FIG. H-7.7



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test data.

FIG. H-7.8

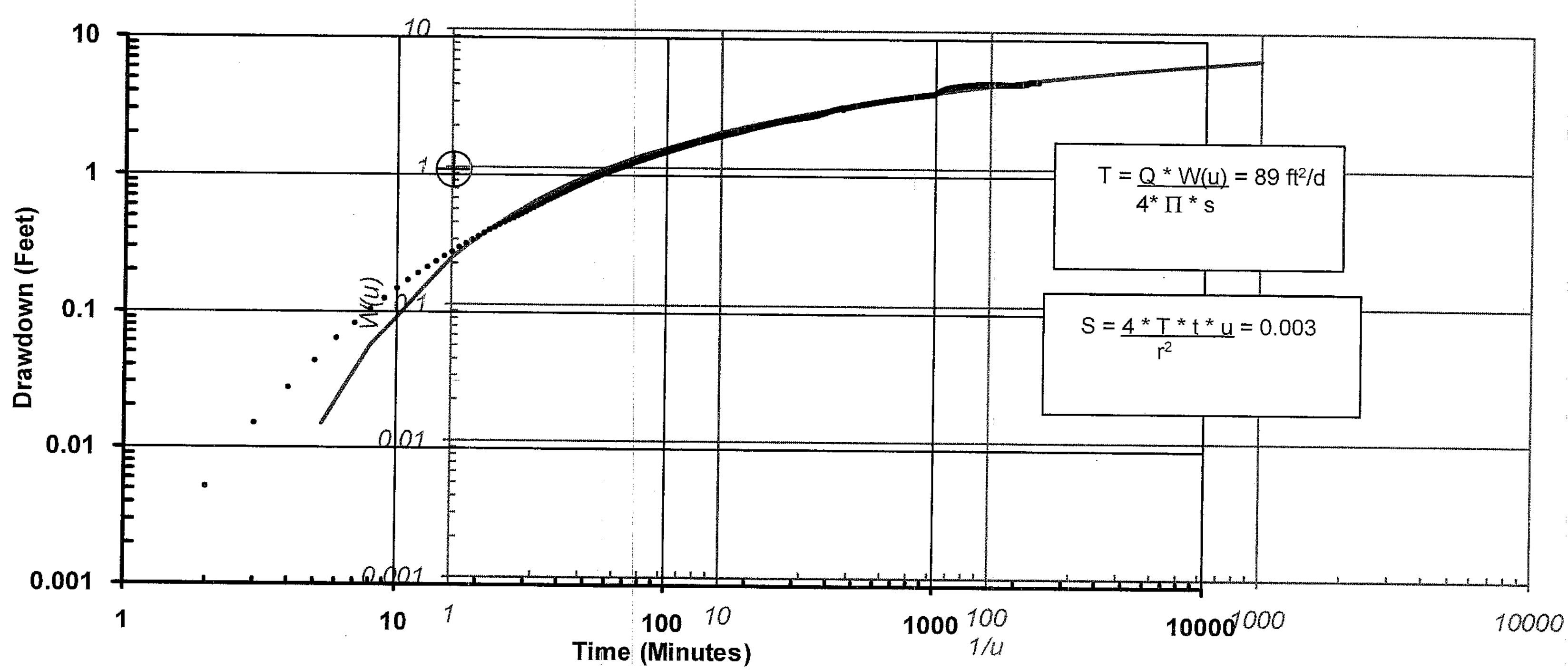
SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 2
MW-6-10 SHALLOW VWP
COOPER-JACOB ANALYSIS

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-7.8



● Monitoring Well MW-1-10 Shallow Drawdown Data

— Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 16.1 min

s = 1.2 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

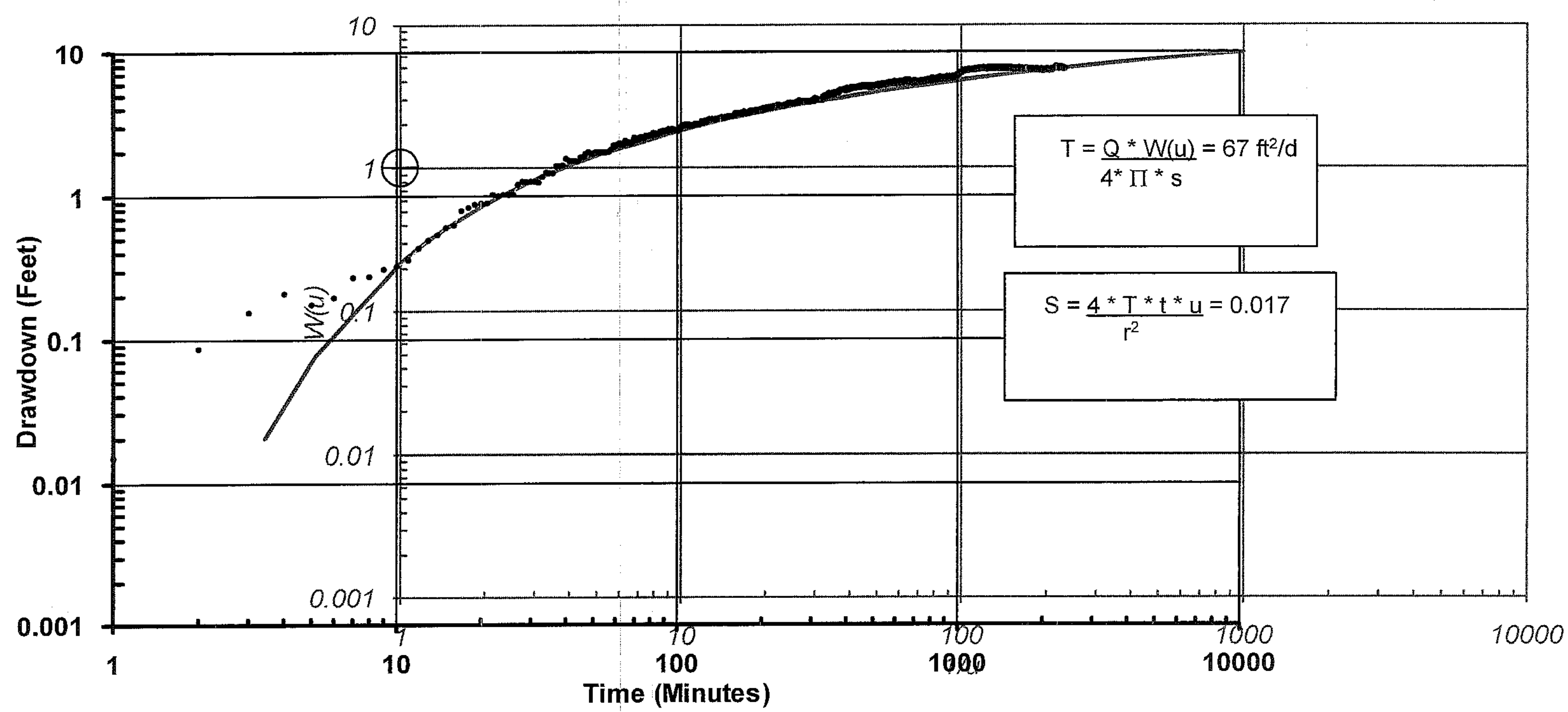
PW-3-10 SHALLOW PUMPING TEST 2
MW-1-10 SHALLOW WELL
THEIS CURVE MATCH

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-7.9

FIG. H-7.9



● Monitoring Well MW-2-10 Shallow VWP Drawdown Data

— Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 10.4 min

s = 1.6 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

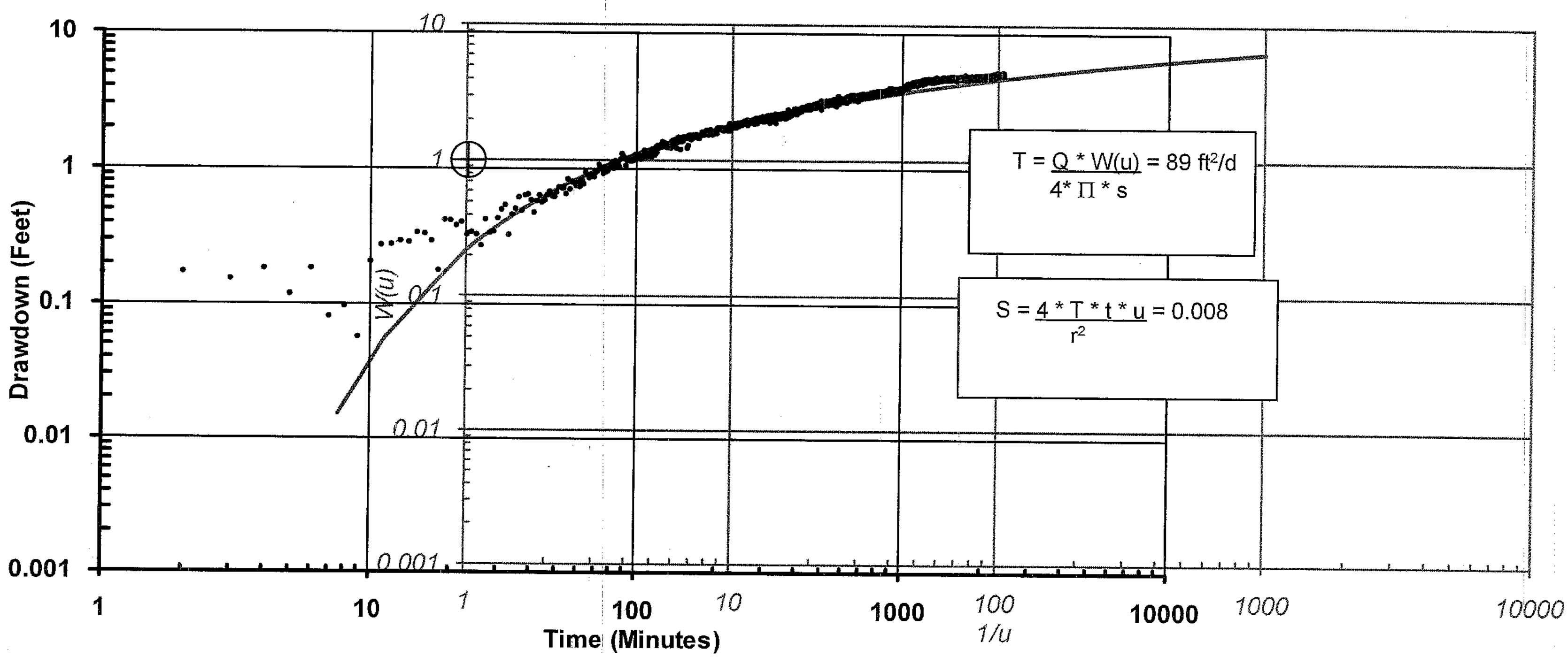
PW-3-10 SHALLOW PUMPING TEST 2
MW-2-10 SHALLOW VWP
THEIS CURVE MATCH

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-7.10

FIG. H-7.10



- Monitoring Well MW-3-10 Shallow VWP Drawdown Data
- Theis Curve
- Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 22.7 min

s = 1.2 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

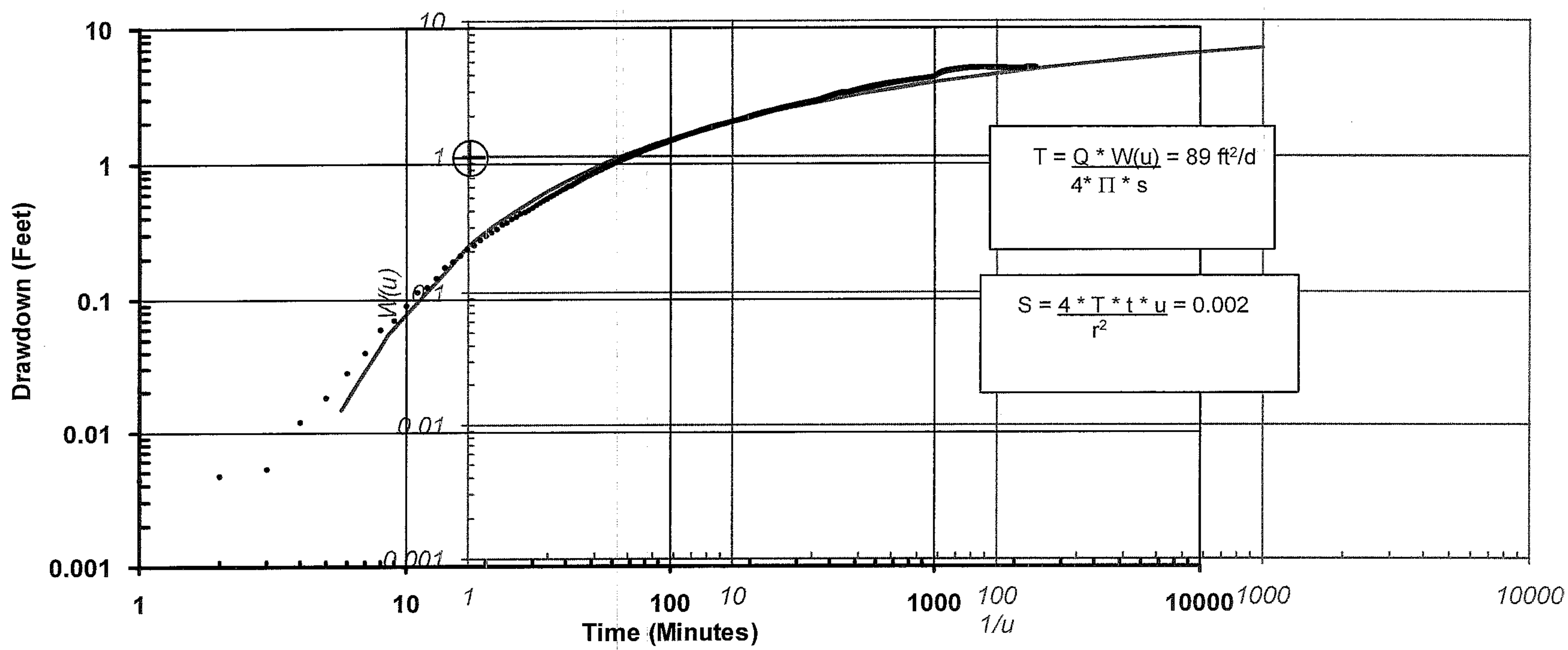
PW-3-10 SHALLOW PUMPING TEST 2
MW-3-10 SHALLOW VWP
THEIS CURVE MATCH

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-7.11

FIG. H-7.11



● Monitoring Well MW-4-10 Shallow Drawdown Data

— Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 17.2 min

s = 1.2 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 2
MW-4-10 SHALLOW WELL
THEIS CURVE MATCH

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
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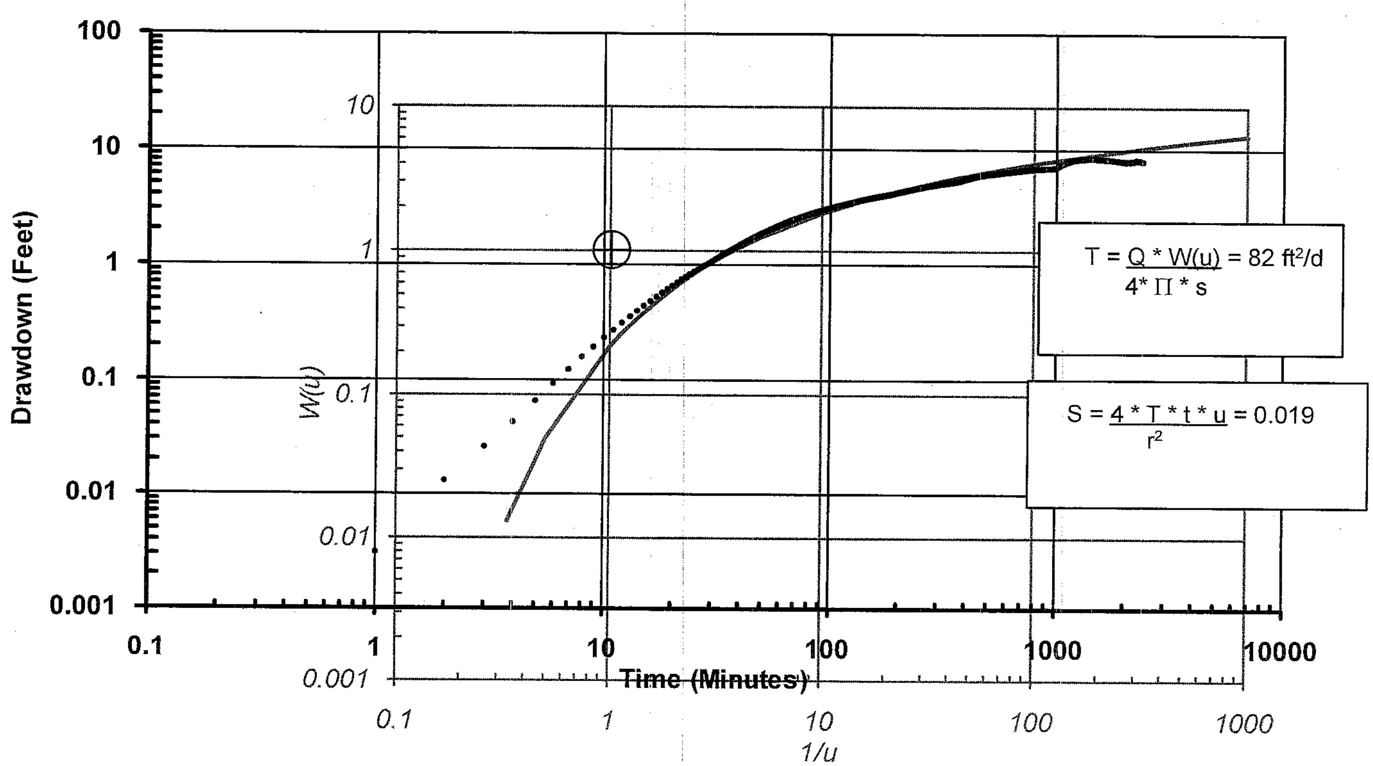
FIG. H-7.12

FIG. H-7.12



● Monitoring Well MW-5-10 Shallow Drawdown Data

— Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 10.7 min

s = 1.3 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

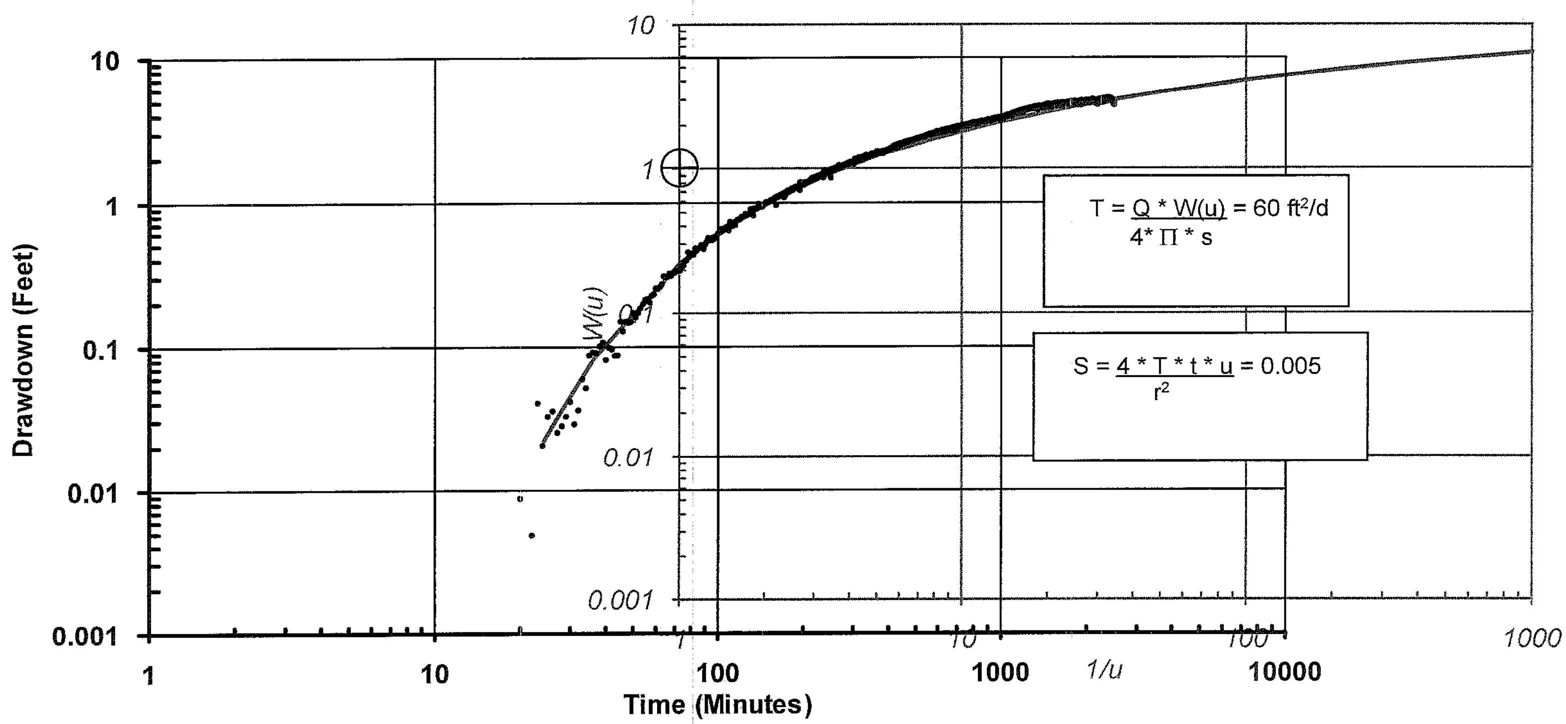
PW-3-10 SHALLOW PUMPING TEST 2
MW-5-10 SHALLOW WELL
THEIS CURVE MATCH

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-7.13

FIG. H-7.13



● Monitoring Well MW-6-10 Shallow VWP Drawdown Data

— Theis Curve

⊕ Match Point

NOTE: See Report for discussion of the Theis Curve Matching Method for analyzing pumping test data.

Match Point

W(u) = 1

1/u = 1

t = 72.5 min

s = 1.8 ft

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 2
MW-6-10 SHALLOW VWP
THEIS CURVE MATCH

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SHANNON & WILSON, INC.
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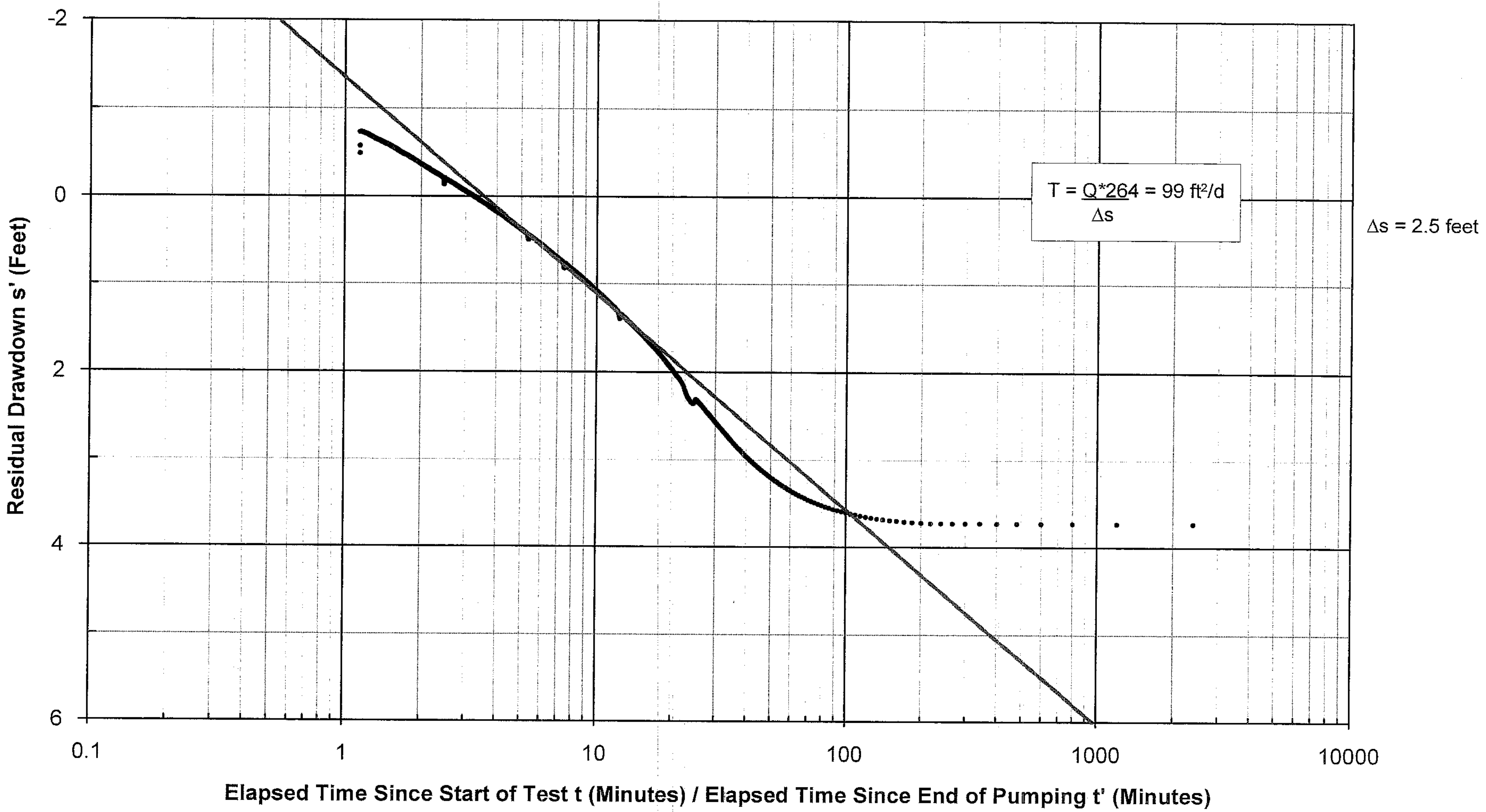
FIG. H-7.14

FIG. H-7.14



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 2
MW-1-10 SHALLOW WELL
RECOVERY DATA

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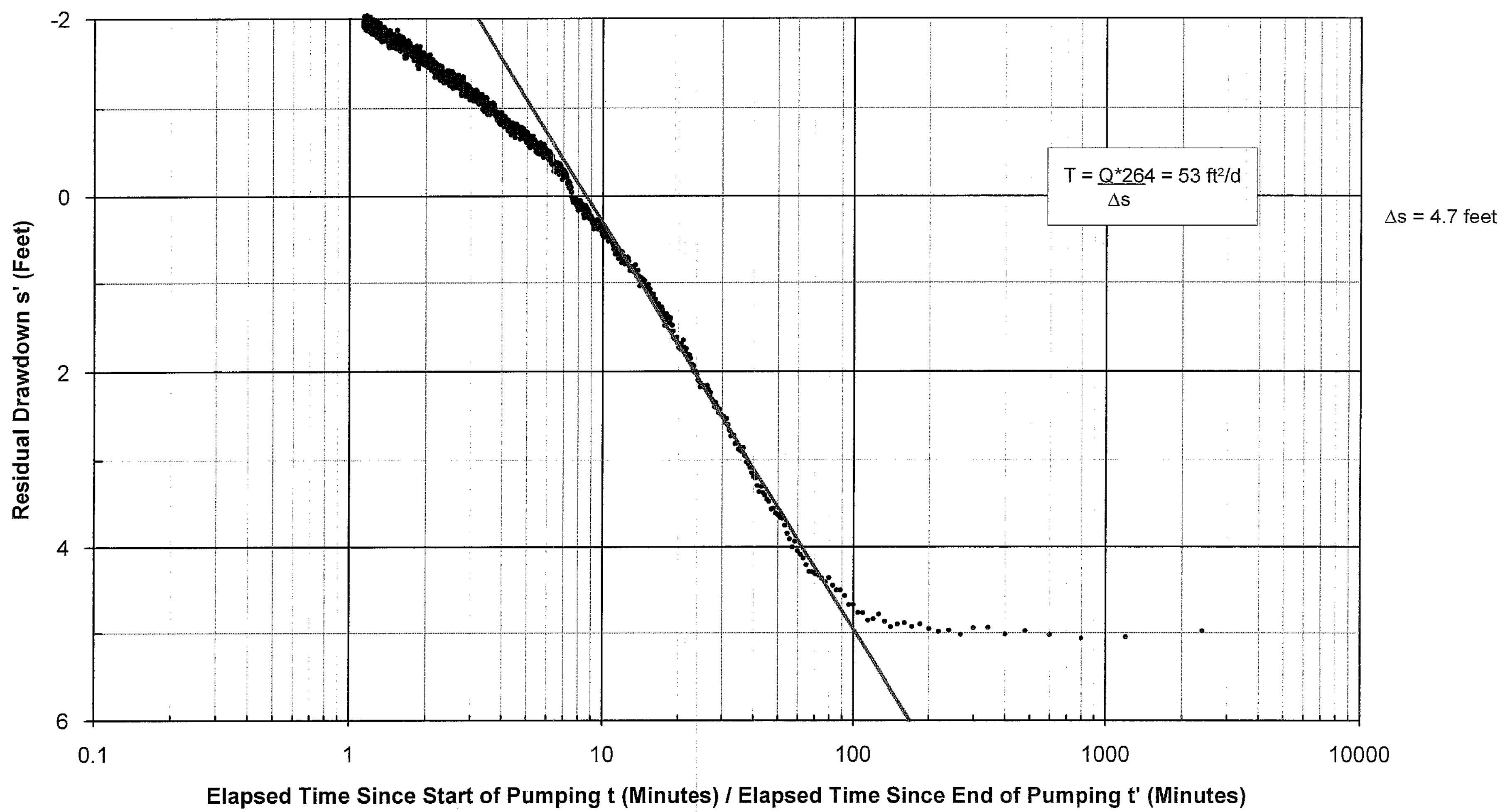
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-7.15

FIG. H-7.15



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 2
MW-2-10 SHALLOW VWP
RECOVERY DATA

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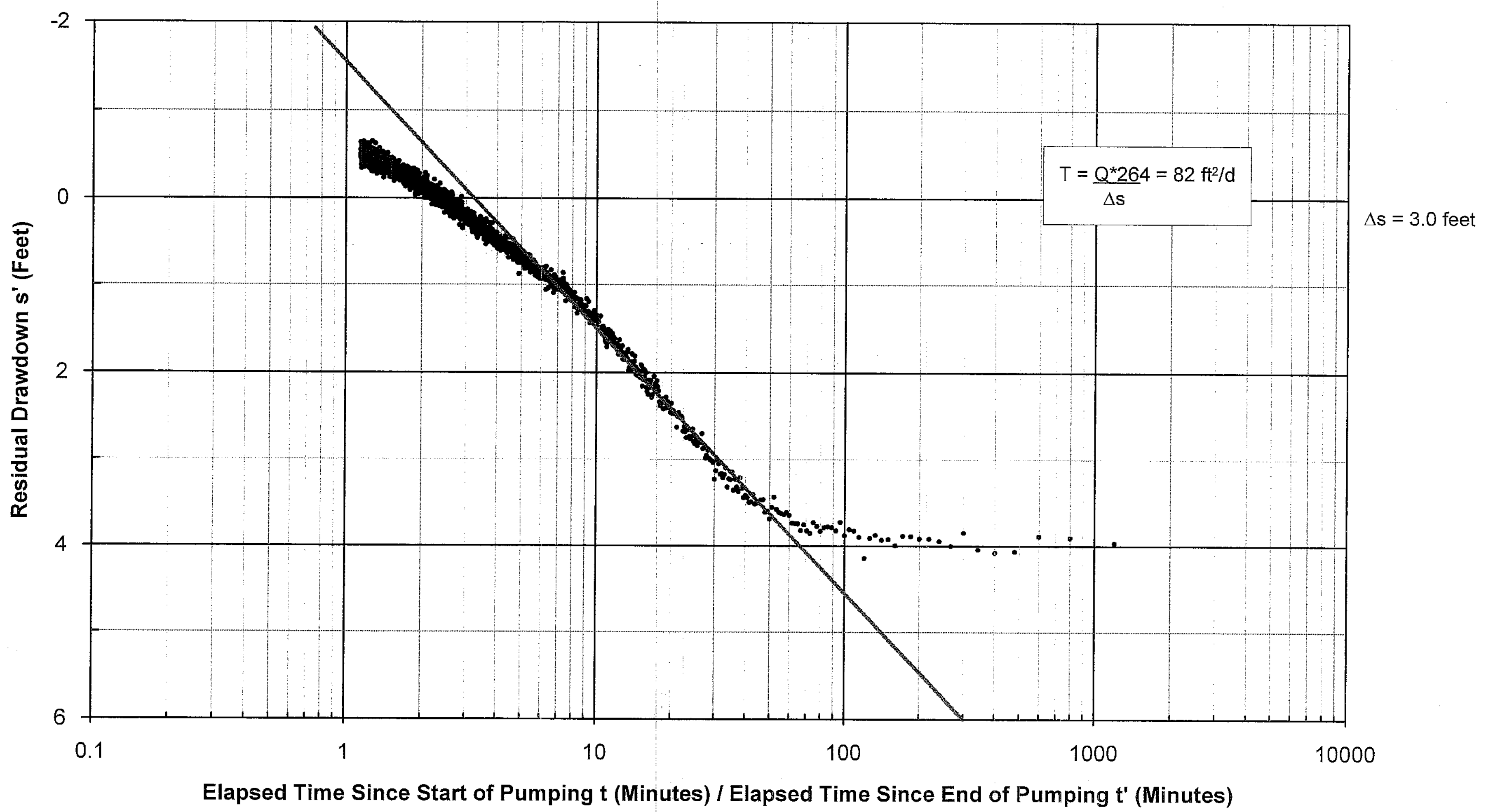
FIG. H-7.16

FIG. H-7.16



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 2
MW-3-10 SHALLOW VWP
RECOVERY DATA

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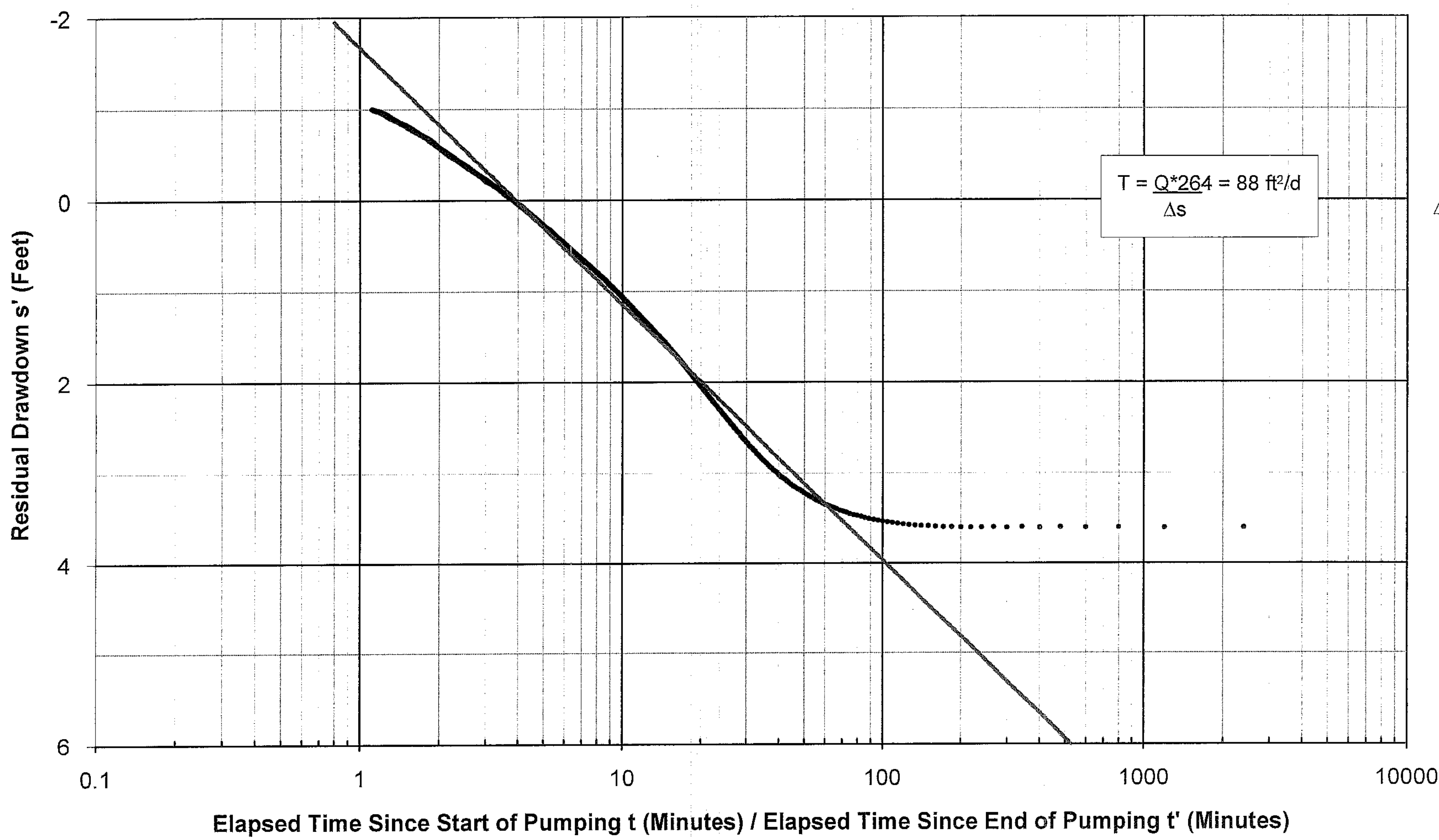
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-7.17

FIG. H-7.17



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 2
MW-4-10 SHALLOW WELL
RECOVERY DATA

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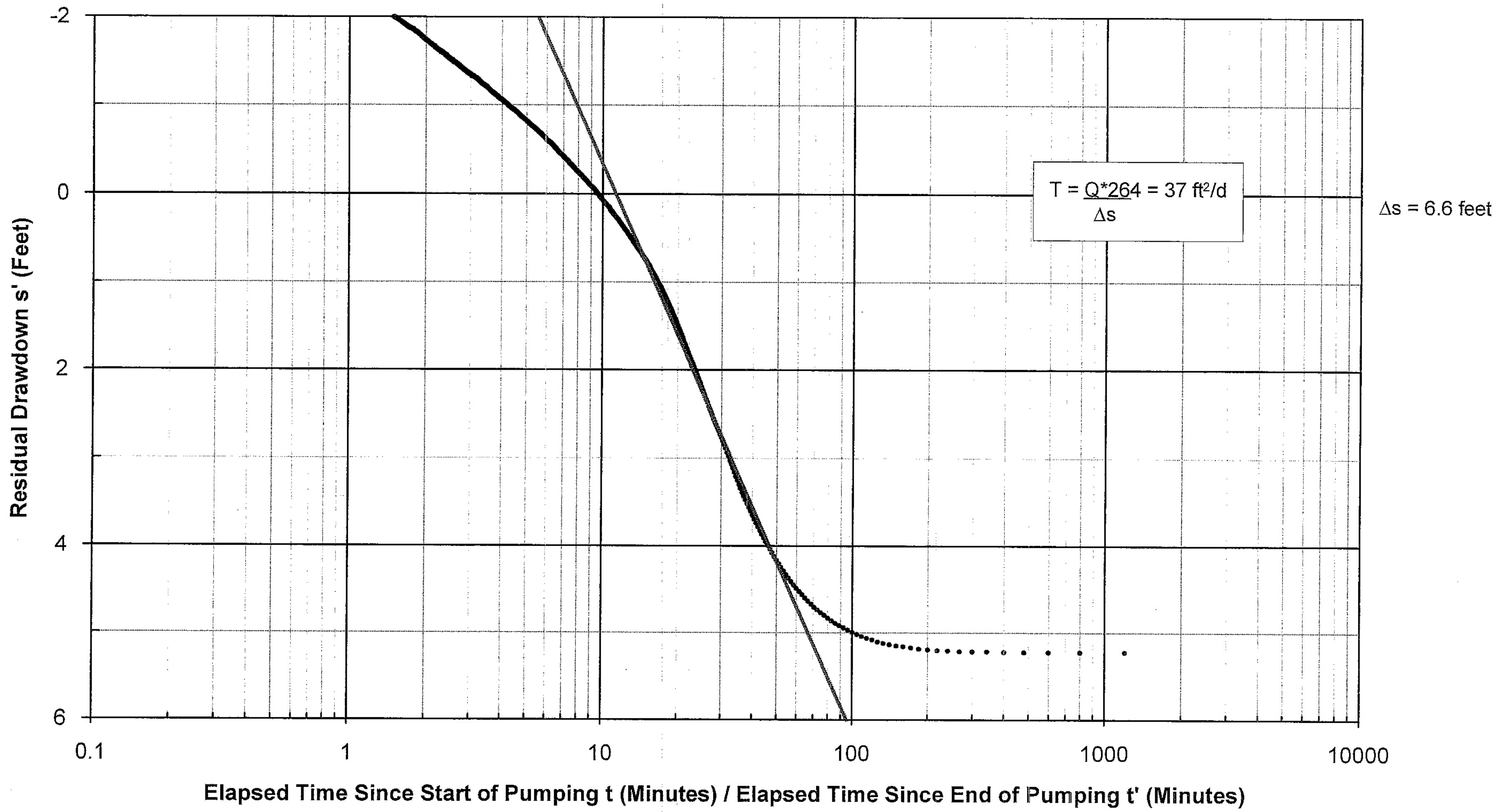
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-7.18

FIG. H-7.18



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

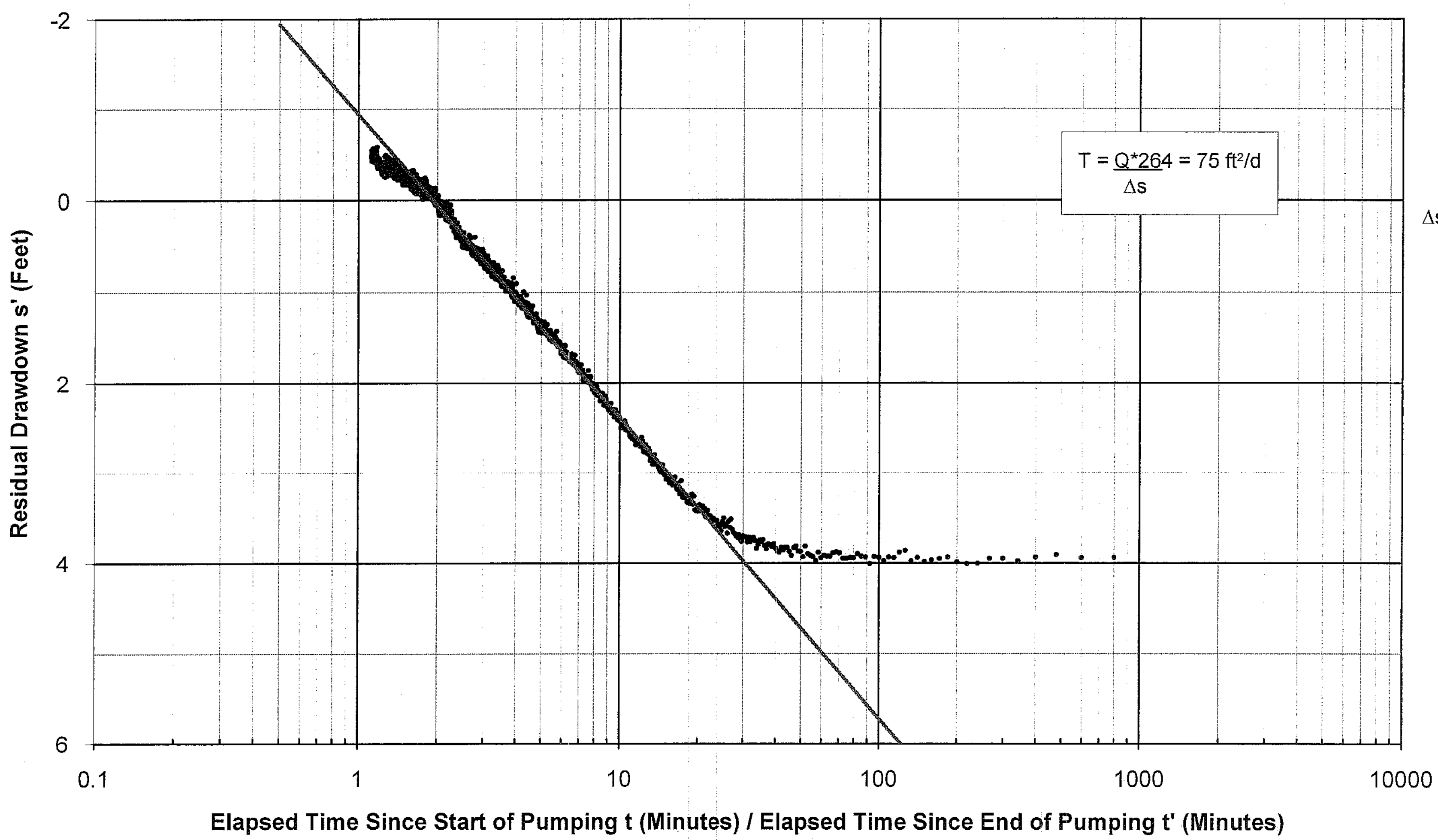
PW-3-10 SHALLOW PUMPING TEST 2
MW-5-10 SHALLOW WELL
RECOVERY DATA

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-7.19

FIG. H-7.19



NOTE: See Report for discussion of the Cooper-Jacob Straight Line Method for analyzing pumping test recovery data.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

PW-3-10 SHALLOW PUMPING TEST 2
MW-6-10 SHALLOW VWP
RECOVERY DATA

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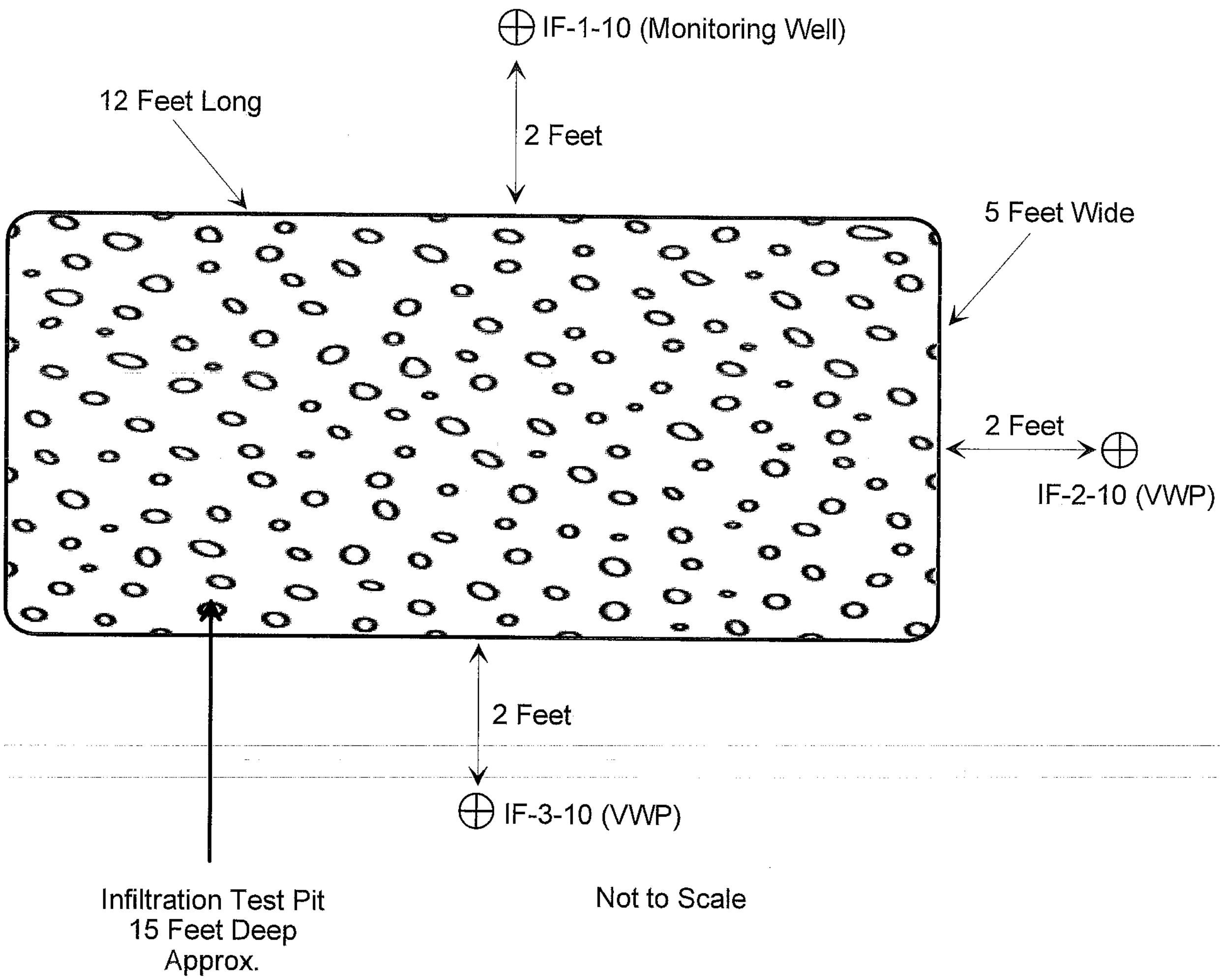
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-7.20

FIG. H-7.20



SR 520 Pontoon Casting Facility
Aberdeen, Washington

INFILTRATION TEST LAYOUT PLAN

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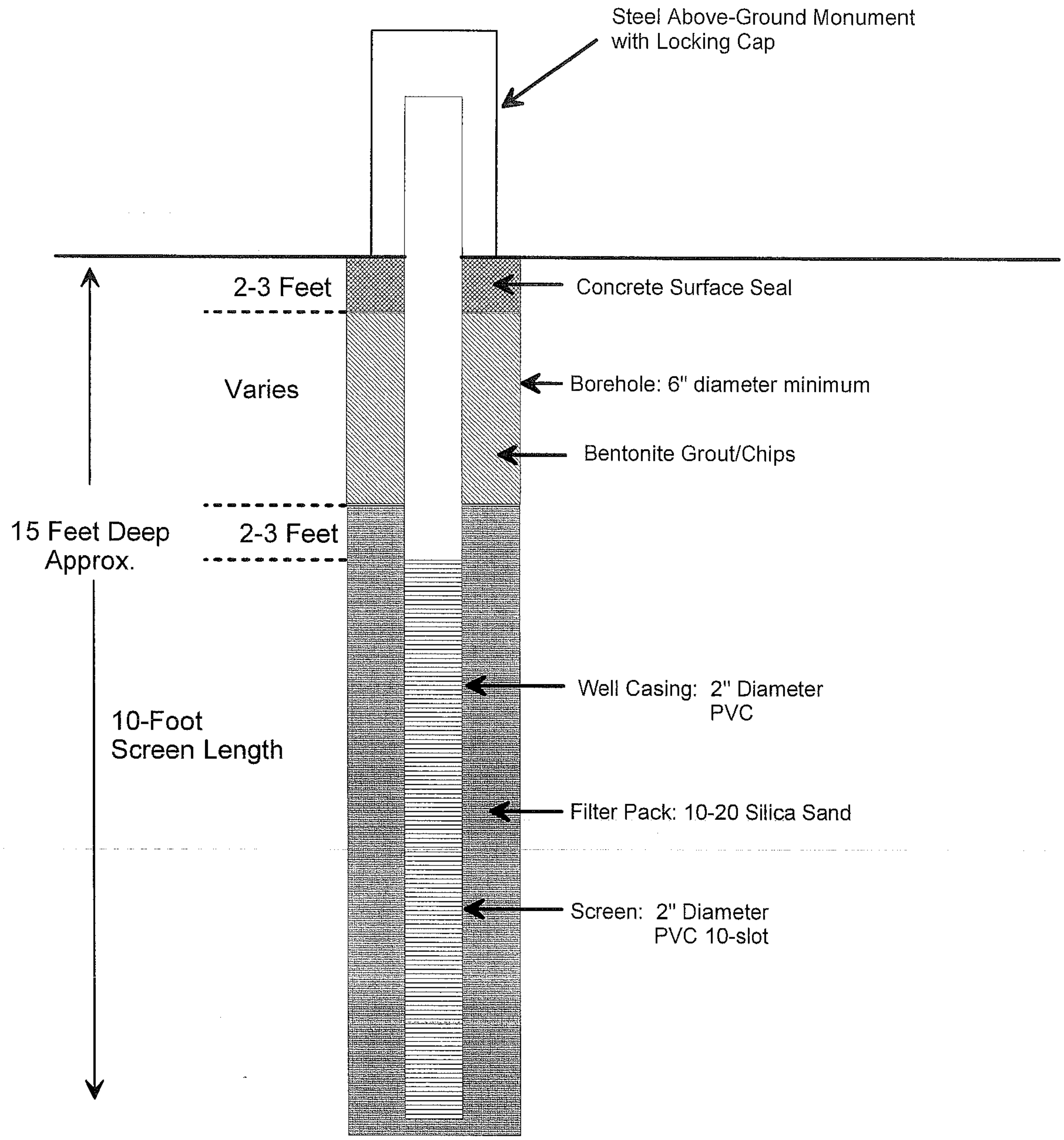
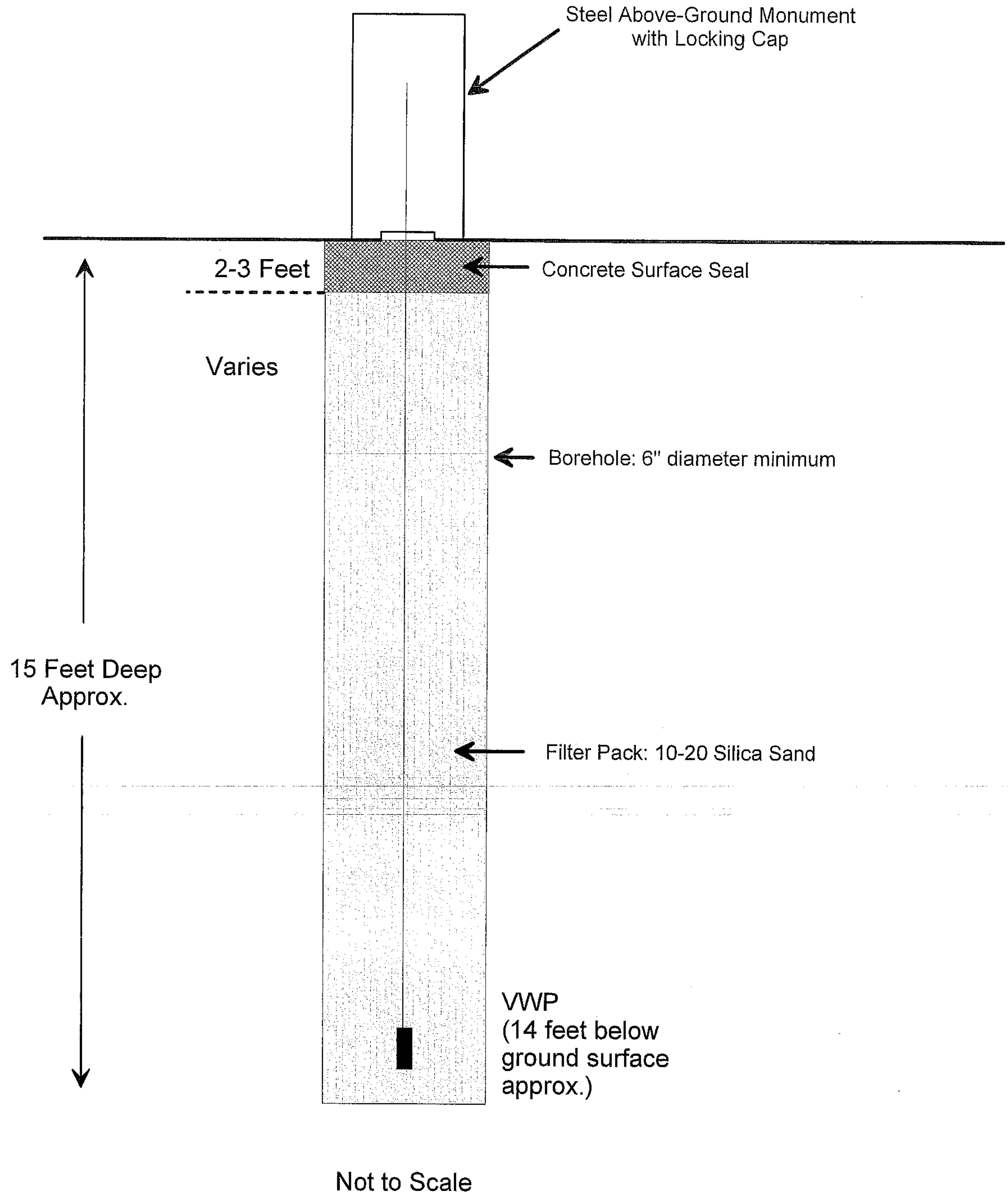
FIG. H-8



Not to Scale

SR 520 Pontoon Casting Facility Aberdeen, Washington	
TYPICAL INFILTRATION TEST MONITORING WELL SCHEMATIC IF-1-10	
September 2010	21-1-21190-014
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. H-9**



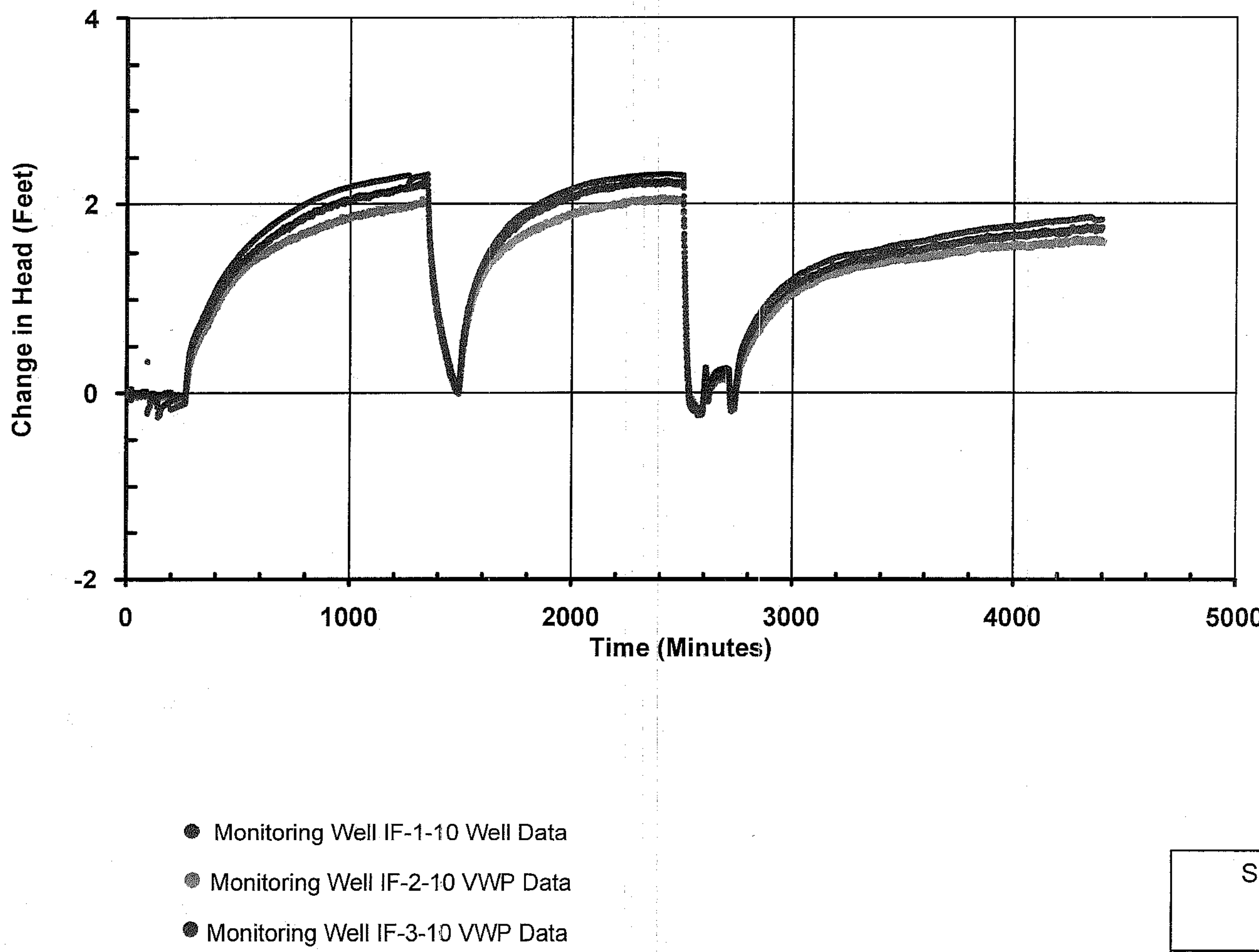
SR 520 Pontoon Casting Facility
Aberdeen, Washington

TYPICAL INFILTRATION TEST
VWP SCHEMATIC
IF-2-10 and IF-3-10

September 2010 | 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-10



SR 520 Pontoon Casting Facility
Aberdeen, Washington

WATER LEVEL HYDROGRAPH
INFILTRATION TESTS

September 2010 — 21-1-21190-014

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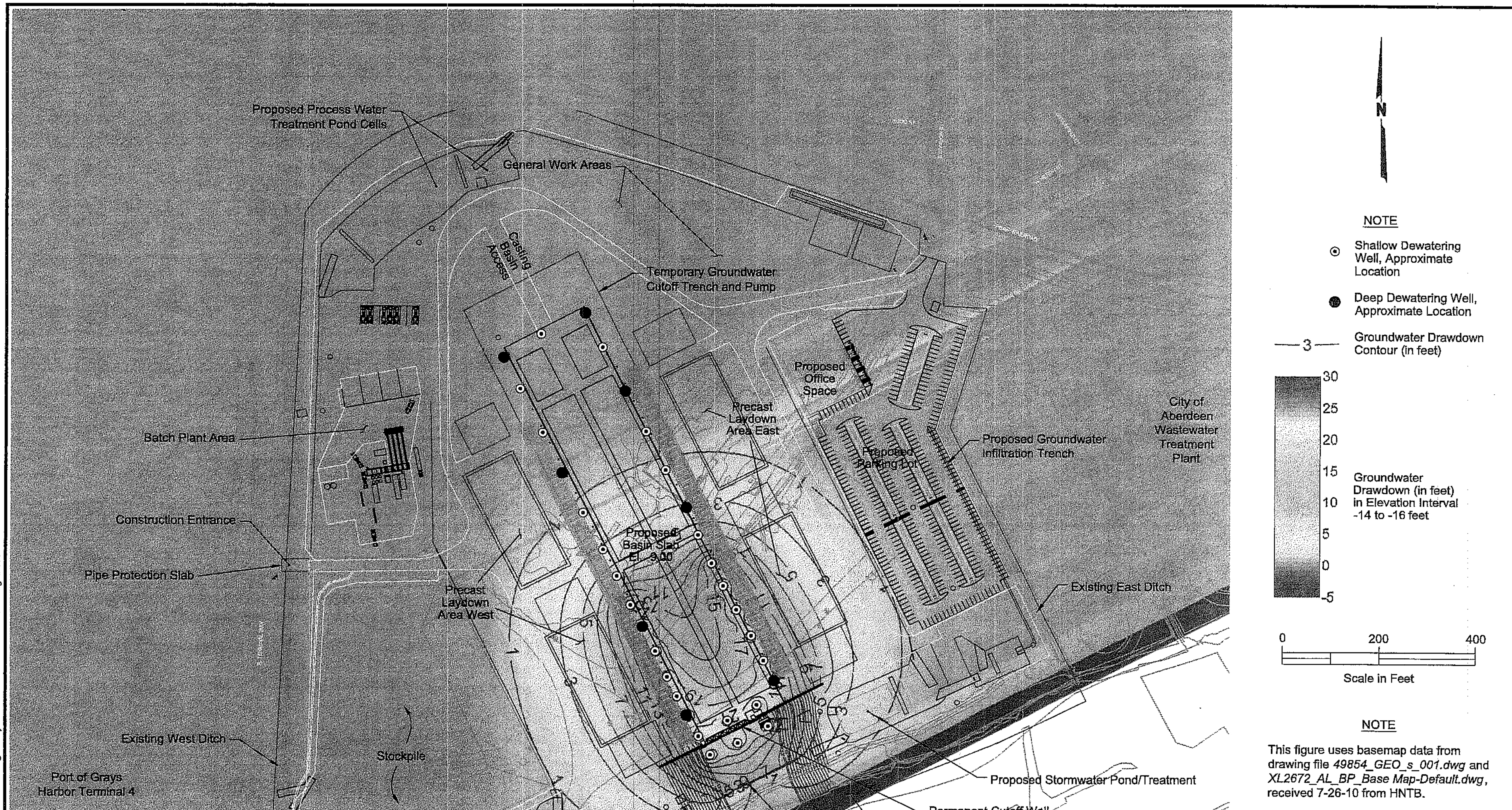
FIG. H-11

FIG. H-11



6 Total Days of Dewatering			
Distance in Basin from Gate (Feet)	Number of Dewatering Wells Pumping	Duration of Pumping (Days)	Minimum Drawdown (Feet)
0 - 230	14	6	14
230 - 460	4	6	4
460 - 690	0	0	0
690 - 920	0	0	0

NOTE

This figure uses basemap data from drawing file *49854_GEO_s_001.dwg* and *XL2672_AL_BP_Base Map-Default.dwg*, received 7-26-10 from HNTB.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

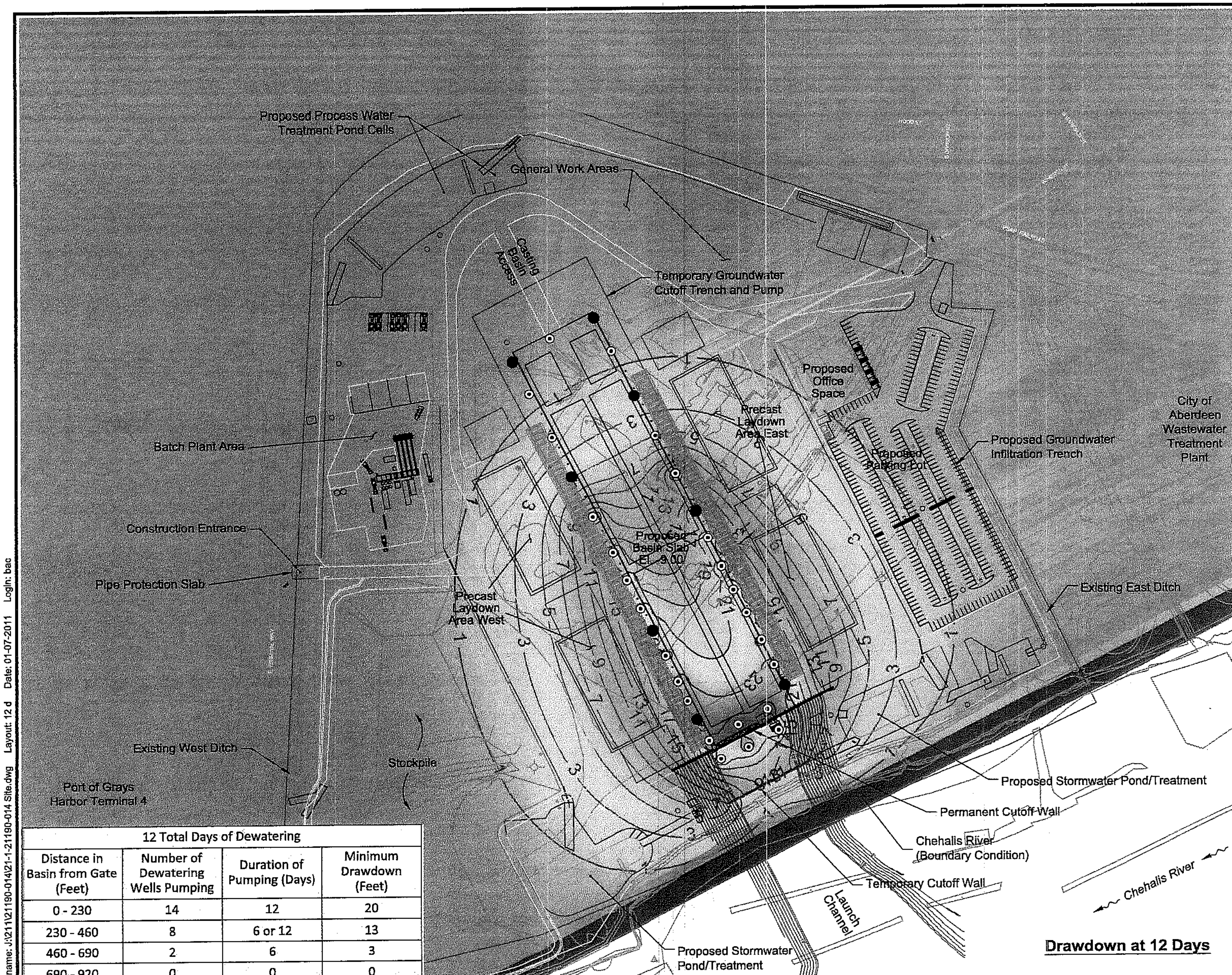
CONSTRUCTION DEWATERING GROUNDWATER DRAWDOWN CONTOUR PLAN

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-12
Sheet 1 of 8

Filename: J:\211\21190-014\21-1-21190-014 Site.dwg Layout: 6 d Date: 01-07-2011 Login: bac



12 Total Days of Dewatering			
Distance in Basin from Gate (Feet)	Number of Dewatering Wells Pumping	Duration of Pumping (Days)	Minimum Drawdown (Feet)
0 - 230	14	12	20
230 - 460	8	6 or 12	13
460 - 690	2	6	3
690 - 920	0	0	0

N

NOTE

Shallow Dewatering Well, Approximate Location

Deep Dewatering Well, Approximate Location

—3— Groundwater Drawdown Contour (in feet)

30, 25, 20, 15, 10, 5, 0, -5

Groundwater Drawdown (in feet) in Elevation Interval -14 to -16 feet

0 200 400

Scale in Feet

NOTE

This figure uses basemap data from drawing file *49854_GEO_s_001.dwg* and *XL2672_AL_BP_Base Map-Default.dwg*, received 7-26-10 from HNTB.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

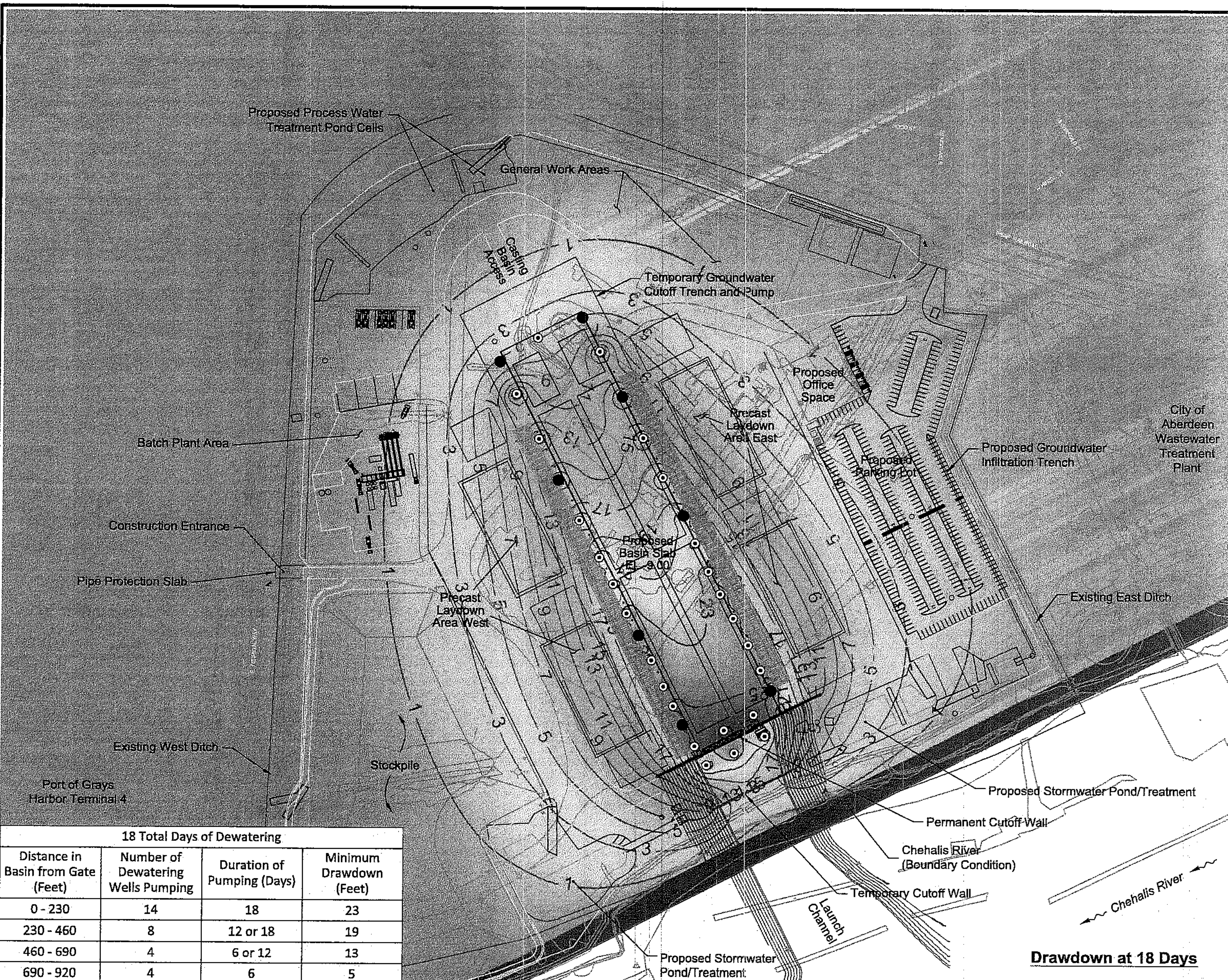
CONSTRUCTION DEWATERING GROUNDWATER DRAWDOWN CONTOUR PLAN

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-12
Sheet 2 of 8

Filename: J:\211\21190-014\21-1-21190-014 Site.dwg Layout: 12 d Date: 01-07-2011 Login: bac



18 Total Days of Dewatering			
Distance in Basin from Gate (Feet)	Number of Dewatering Wells Pumping	Duration of Pumping (Days)	Minimum Drawdown (Feet)
0 - 230	14	18	23
230 - 460	8	12 or 18	19
460 - 690	4	6 or 12	13
690 - 920	4	6	5

Drawdown at 18 Days

N

NOTE

- Shallow Dewatering Well, Approximate Location
- Deep Dewatering Well, Approximate Location
- —3— Groundwater Drawdown Contour (in feet)

30, 25, 20, 15, 10, 5, 0, -5 — Groundwater Drawdown (in feet) in Elevation Interval -14 to -16 feet

0 200 400

Scale in Feet

NOTE

This figure uses basemap data from drawing file *49854_GEO_s_001.dwg* and *XL2672_AL_BP_Base Map-Default.dwg*, received 7-26-10 from HNTB.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

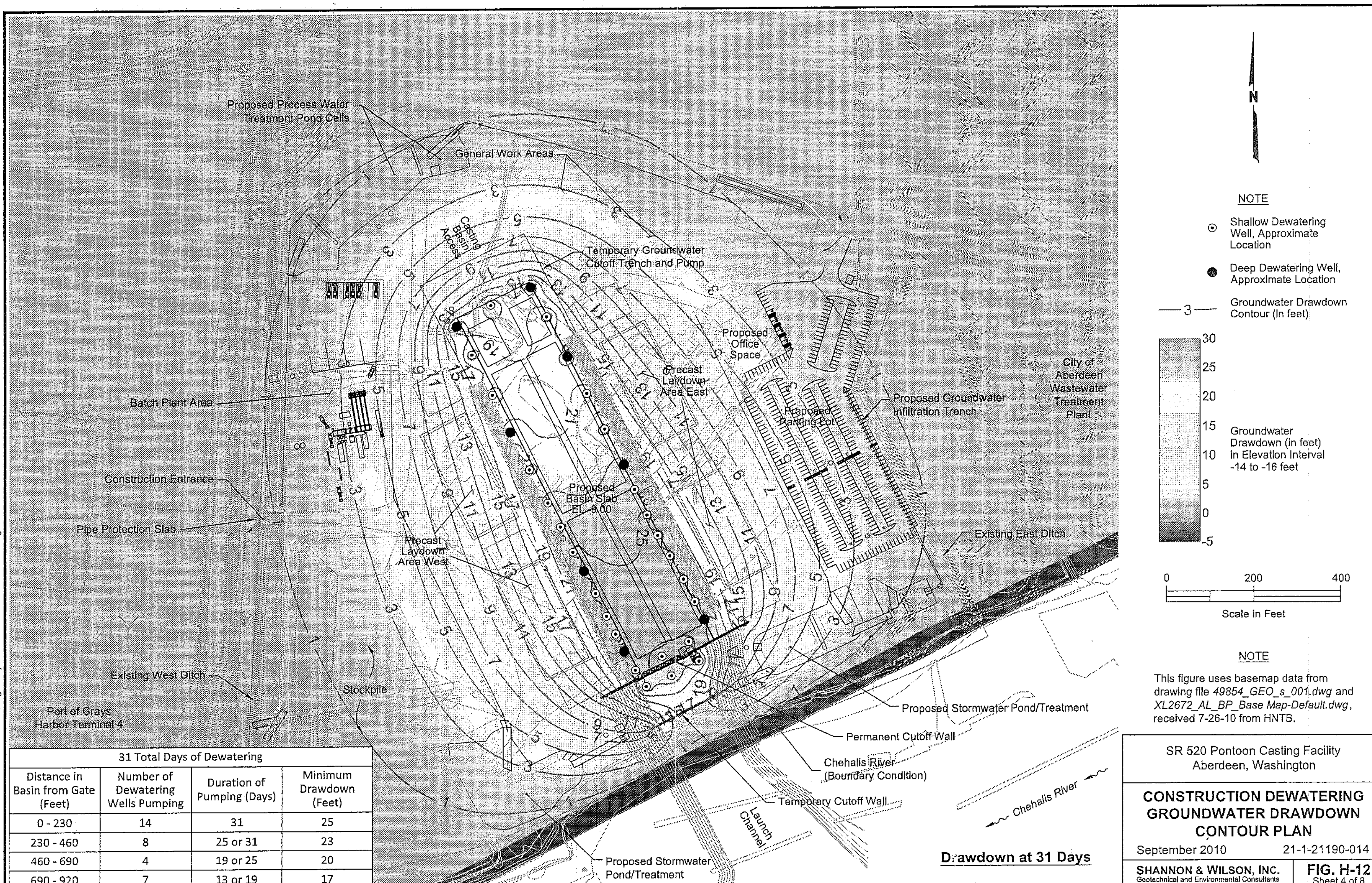
CONSTRUCTION DEWATERING GROUNDWATER DRAWDOWN CONTOUR PLAN

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-12
Sheet 3 of 8

Filename: J:\211\21190-014\21-1-21190-014 Site.dwg Layout: 18 d Date: 01-07-2011 Login: bac



31 Total Days of Dewatering			
Distance in Basin from Gate (Feet)	Number of Dewatering Wells Pumping	Duration of Pumping (Days)	Minimum Drawdown (Feet)
0 - 230	14	31	25
230 - 460	8	25 or 31	23
460 - 690	4	19 or 25	20
690 - 920	7	13 or 19	17

Drawdown at 31 Days

NOTE

This figure uses basemap data from drawing file *49854_GEO_s_001.dwg* and *XL2672_AL_BP_Base Map-Default.dwg*, received 7-26-10 from HNTB.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

CONSTRUCTION DEWATERING GROUNDWATER DRAWDOWN CONTOUR PLAN

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
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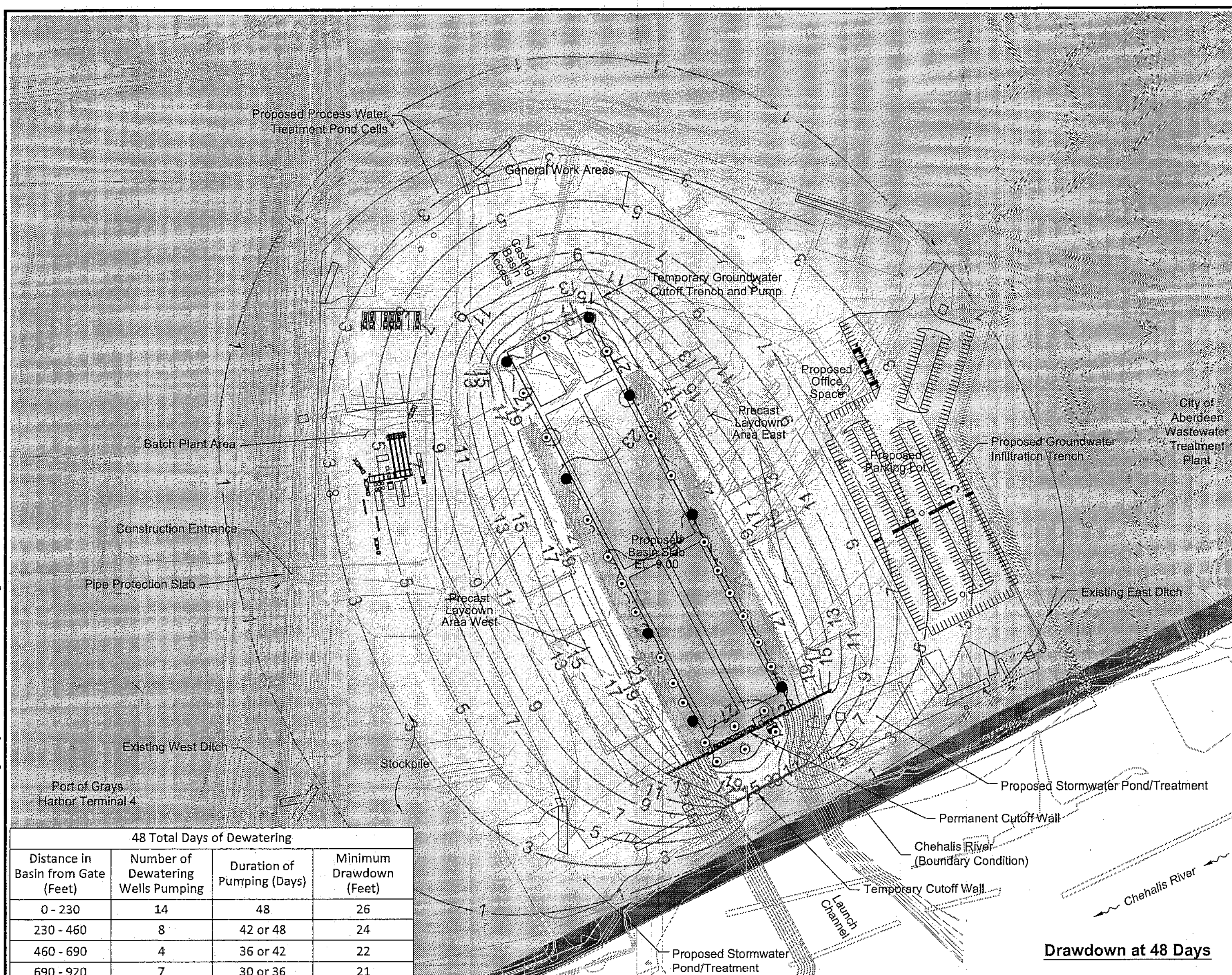
FIG. H-12
Sheet 4 of 8

Filename: J:\211\21190-014\21-1-21190-014 Site.dwg Layout: 31 d Date: 01-07-2011 Login: bac



48 Total Days of Dewatering			
Distance in Basin from Gate (Feet)	Number of Dewatering Wells Pumping	Duration of Pumping (Days)	Minimum Drawdown (Feet)
0 - 230	14	48	26
230 - 460	8	42 or 48	24
460 - 690	4	36 or 42	22
690 - 920	7	30 or 36	21

NOTE

- Shallow Dewatering Well, Approximate Location
- Deep Dewatering Well, Approximate Location
- 3 — Groundwater Drawdown Contour (in feet)

30, 25, 20, 15, 10, 5, 0, -5 — Groundwater Drawdown (in feet) in Elevation Interval -14 to -16 feet

0 200 400
Scale in Feet

NOTE

This figure uses basemap data from drawing file *49854_GEO_s_001.dwg* and *XL2672_AL_BP_Base Map-Default.dwg*, received 7-26-10 from HNTB.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

CONSTRUCTION DEWATERING GROUNDWATER DRAWDOWN CONTOUR PLAN

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
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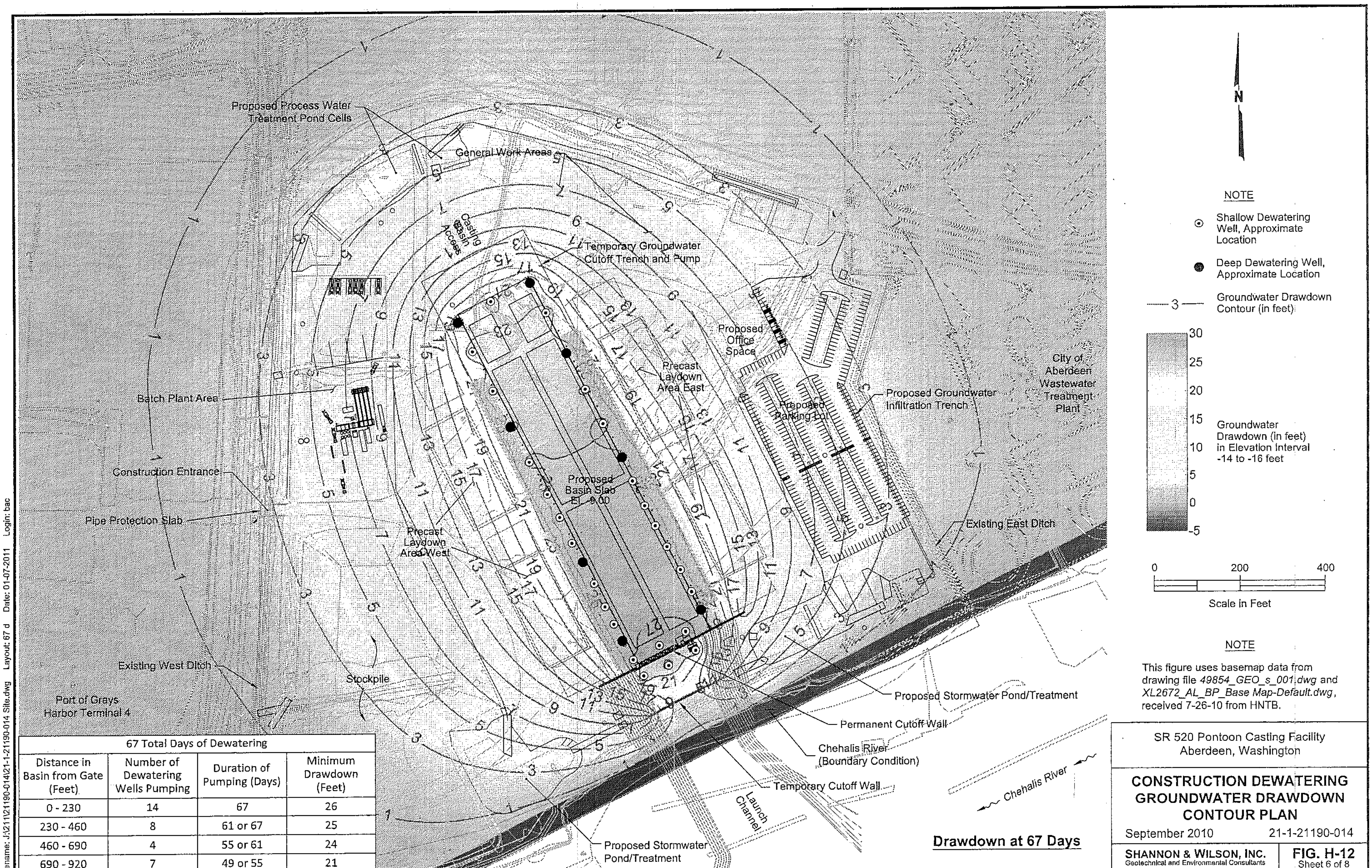
FIG. H-12
Sheet 5 of 8



67 Total Days of Dewatering			
Distance in Basin from Gate (Feet)	Number of Dewatering Wells Pumping	Duration of Pumping (Days)	Minimum Drawdown (Feet)
0 - 230	14	67	26
230 - 460	8	61 or 67	25
460 - 690	4	55 or 61	24
690 - 920	7	49 or 55	21

NOTE

This figure uses basemap data from drawing file *49854_GEO_s_001.dwg* and *XL2672_AL_BP_Base Map-Default.dwg*, received 7-26-10 from HNTB.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

CONSTRUCTION DEWATERING GROUNDWATER DRAWDOWN CONTOUR PLAN

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
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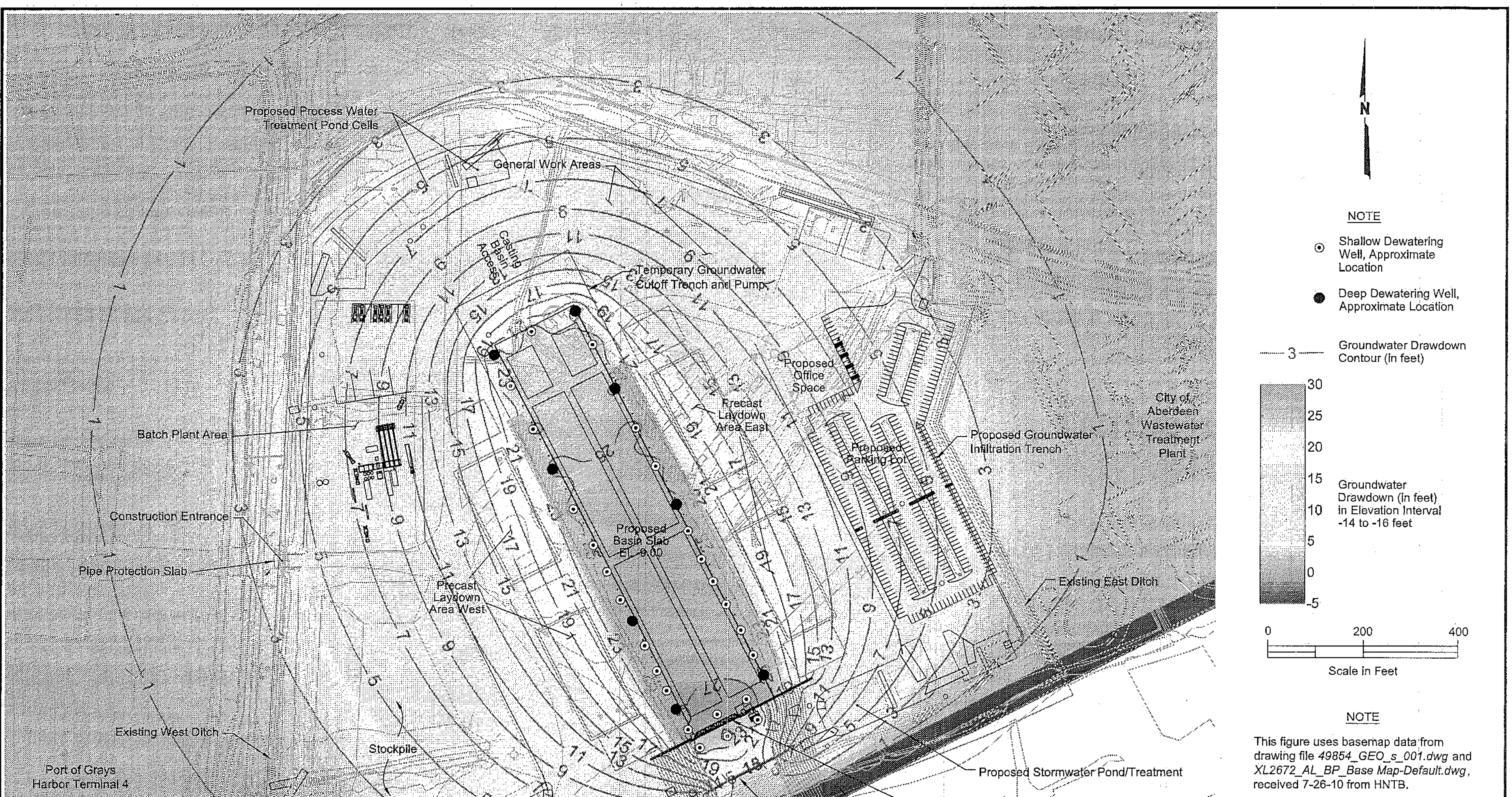
FIG. H-12
Sheet 6 of 8

Filename: J:\211\21190-014\21-1-21190-014 Site.dwg Layout: 67 d Date: 01-07-2011 Login: bac



90 Total Days of Dewatering			
Distance in Basin from Gate (Feet)	Number of Dewatering Wells Pumping	Duration of Pumping (Days)	Minimum Drawdown (Feet)
0 - 230	14	90	26
230 - 460	8	84 or 90	25
460 - 690	4	78 or 84	24
690 - 920	7	72 or 78	23

NOTE

This figure uses basemap data from drawing file *49854_GEO_s_001.dwg* and *XL2672_AL_BP_Base Map-Default.dwg*, received 7-26-10 from HNTB.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

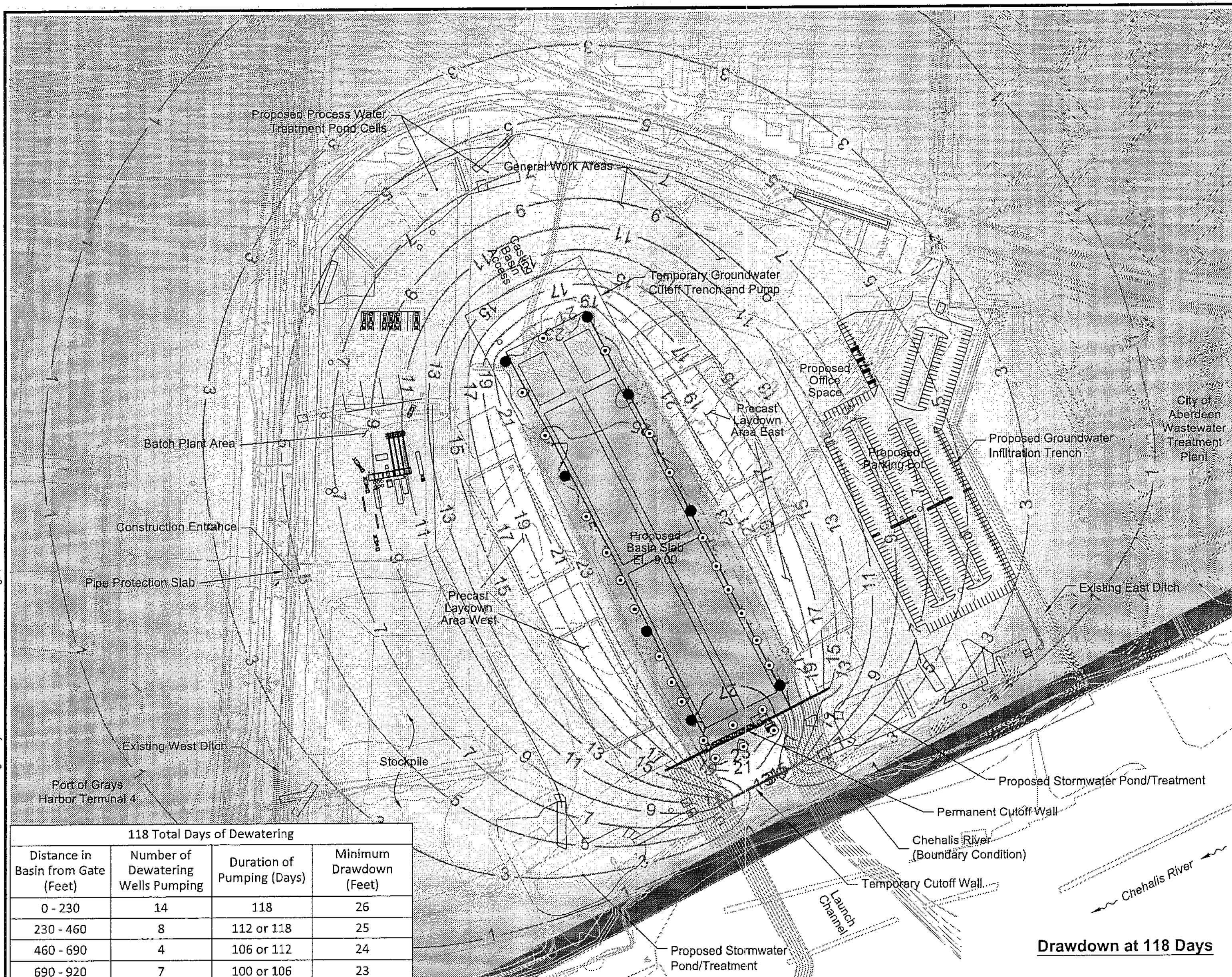
CONSTRUCTION DEWATERING GROUNDWATER DRAWDOWN CONTOUR PLAN

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-12
Sheet 7 of 8

Filename: J:\211\21190-014\21-1-21190-014 Site.dwg Layout: 90 d Date: 01-07-2011 Login: bac



Drawdown at 118 Days

118 Total Days of Dewatering			
Distance in Basin from Gate (Feet)	Number of Dewatering Wells Pumping	Duration of Pumping (Days)	Minimum Drawdown (Feet)
0 - 230	14	118	26
230 - 460	8	112 or 118	25
460 - 690	4	106 or 112	24
690 - 920	7	100 or 106	23

NOTE

This figure uses basemap data from drawing file *49854_GEO_s_001.dwg* and *XL2672_AL_BP_Base Map-Default.dwg*, received 7-26-10 from HNTB.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

CONSTRUCTION DEWATERING GROUNDWATER DRAWDOWN CONTOUR PLAN

September 2010 21-1-21190-014

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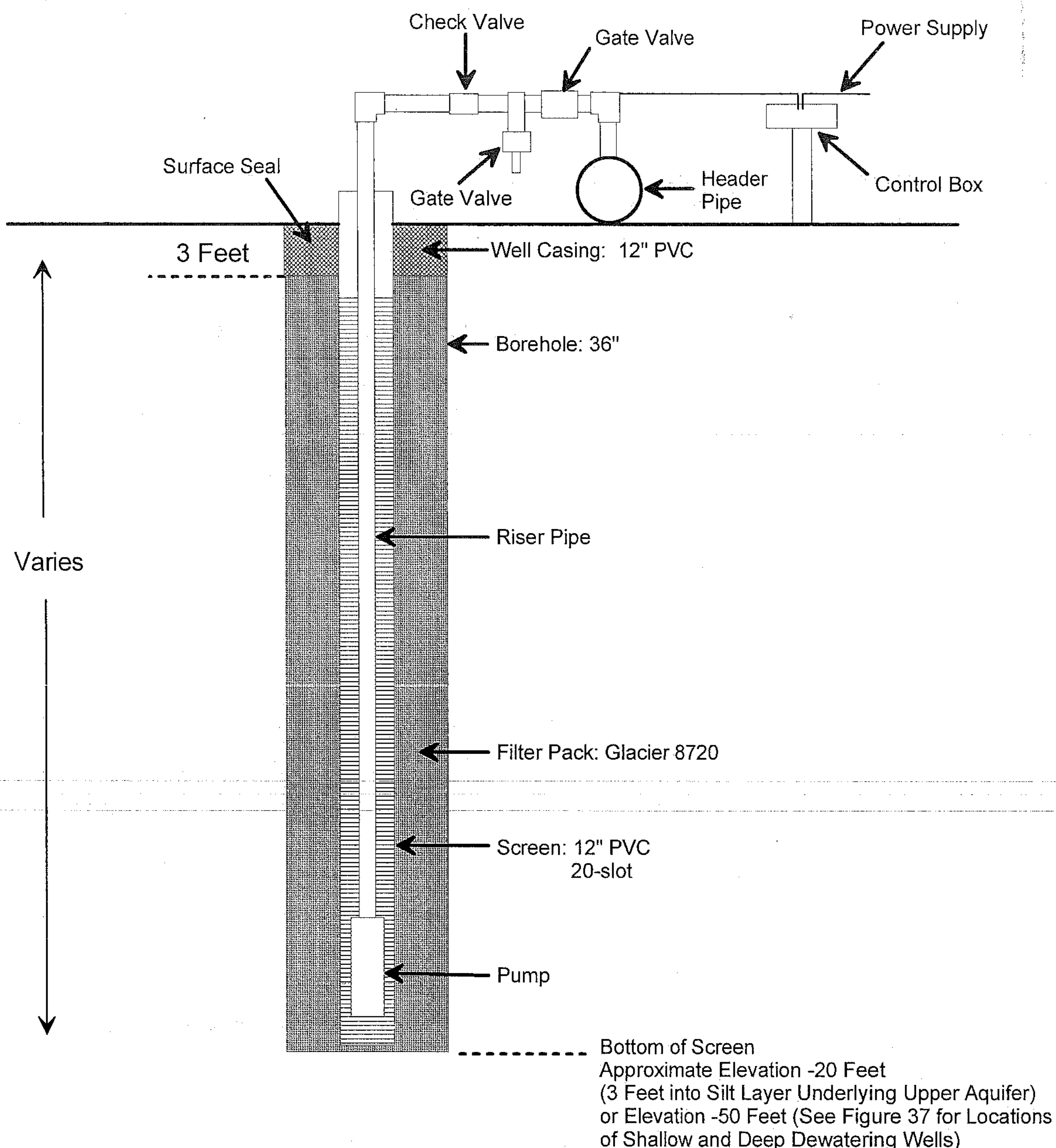
FIG. H-12
Sheet 8 of 8

Filename: J:\211\21190-014\21-1-21190-014 Site.dwg Layout: 118 d Date: 01-07-2011 Login: bac



Not to Scale

SR 520 Pontoon Casting Facility
Aberdeen, Washington

TYPICAL DEWATERING WELL SCHEMATIC

September 2010 21-1-21190-014

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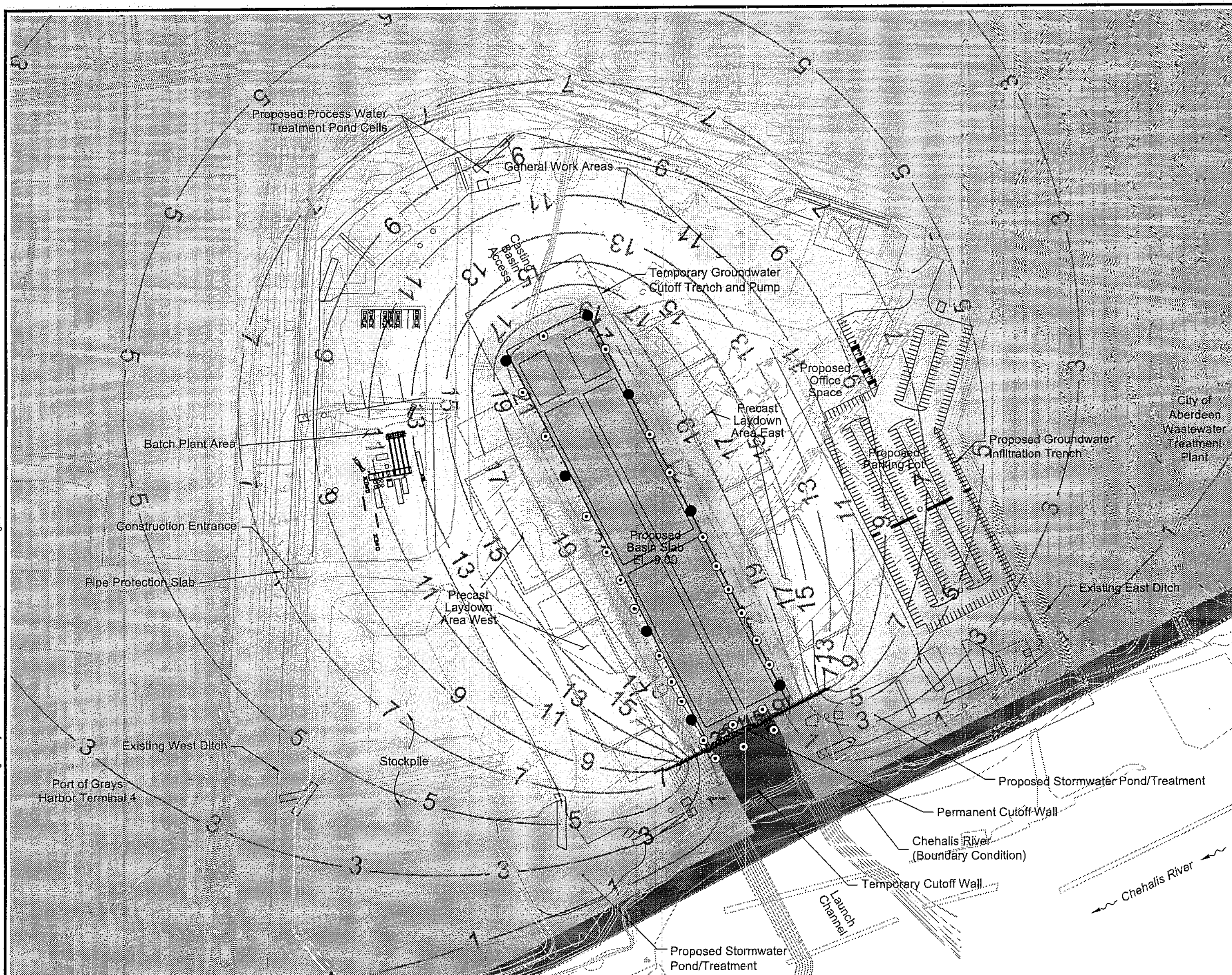
FIG. H-13



N

NOTE

- Shallow Dewatering Well, Approximate Location
- Deep Dewatering Well, Approximate Location
- 3 — Groundwater Drawdown Contour (in feet)

Groundwater Drawdown (in feet) in Elevation Interval -12 to -14 feet

30, 25, 20, 15, 10, 5, 0, -5

0 200 400

Scale in Feet

NOTE

This figure uses basemap data from drawing file *49854_GEO_s_001.dwg* and *XL2672_AL_BP_Base Map-Default.dwg*, received 7-26-10 from HNTB.

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Aberdeen, Washington

PERMANENT DEWATERING GROUNDWATER DRAWDOWN CONTOUR PLAN

September 2010 21-1-21190-014

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. H-14

Filename: J:\211\21190-014\21-1-21190-014 Site.dwg Layout: Perm Dewat Draw Date: 01-07-2011 Login: bac

APPENDIX I

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to and part of Report 21-1-21190-015

Date: February 18, 2011
To: Mr. Tom Schnetzer
HNTB

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

APPENDIX II:
Slope Stability

June 28, 2013

Mr. Will Morgan
Kiewit-General
1301 West Heron Street
Aberdeen, WA 98520

RE: SOUTHEAST CORNER BASIN SLOPE STABILITY, STATE ROUTE (SR) 520 PONTOON CASTING FACILITY, ABERDEEN, WASHINGTON

Dear Mr. Morgan:

This letter provides additional information regarding our assessment of the slope in the southeast corner of the pontoon casting facility. Specifically, this letter includes a discussion of the overexcavation and backfill operations for the portion of the southeast corner basin slope that moved on May 2011, the southeast corner rebuilt slope static factor of safety, and observations of ground surface cracking during equipment loading as related to stability of the rebuilt slope.

During the evening of May 19 or the morning of May 20, 2011, a 40- to 50-foot-long portion of the eastern side slope at the south east corner of the basin moved laterally and vertically about 2 to 4 feet. Letters presenting the cause of the slope instability at the southeast corner of the basin, recommendations regarding future excavation of the basin slopes, and recommendations for backfill of the southeast corner of the basin where the slope instability occurred were submitted on June 9 and August 12, 2011.

OVEREXCAVATION AND BACKFILL OPERATIONS

The slope instability area was overexcavated and backfilled with imported select granular borrow between May 20 and 23, 2011. The approximate depth and transverse extent of the overexcavation are shown on Figure 1. Longitudinally the overexcavation extended between the first and ninth crane trestle piles on the south side of the basin. The imported select borrow was placed in loose lifts and compacted with passes of a 10 ton static roller. In our opinion, the overexcavation and backfill operations were performed in accordance with our recommendations.

REBUILT SLOPE STATIC FACTOR OF SAFETY

The global static stability results considering the overexcavated and backfilled slope repair are shown on Figure 1. Figure 1 provides the global stability using drained strength properties for the site soils and typical surcharge loading. Based on this analysis, the static factor of safety for the overexcavated and backfilled slope repair is greater than 1.3. Our back calculated soil shear strength utilized in the slope stability analysis shown on Figure 1 and in our June 9 and August 12, 2011 letters considered the loading that caused the slope instability and back-calculated reduced soil properties associated with the estimated failure plane. In our opinion, the stability of the basin slopes as designed is appropriate and would provide suitable static factors of safety as required by the WSDOT Geotechnical Design Manual.

GROUND SURFACE CRACKING DURING EQUIPMENT LOADING

Following repair of the southeast corner slope, observations of surface cracking were made east of the southeast corner slope during utilization of a pump truck around June 30, 2011. The surface cracking was noted around the edges of the crane mat beneath the pump truck. The front and rear edges of the crane mat were placed approximately 50 and 75 feet, respectively, from the back of the longitudinal crane trestle beam. The allowable ground pressure from the pump truck outrigger was limited to no more than 500 pounds per square foot (psf) using a grillage of H-beams and a crane mat.

Using the soil shear strength included in our released for construction geotechnical report and back-calculated from the slope instability and the pump truck surcharge loading, we estimate that the factor of safety is greater than 1.3 based on the analysis results shown in Figure 1. That is, the temporary surcharge loading used in our analysis and shown on Figure 1 is greater than what was applied to the ground by the pump truck described above.

The observations of surface cracking were confined around the crane mat edges and were likely associated with surficial compaction of the imported sand and gravel during pump truck loading, in our opinion. Our opinion is based on: site observations of the localized surficial cracking, the relatively large 50 foot distance between crest of the slope and the closest edge of the crane mat, suitable performance of the pump truck at several locations around the pontoon casting basin site that did not result in surficial cracking, and the slope stability analyses that provides a suitable factor of safety while using a surcharge loading greater than the 500 psf loading applied by the pump truck. Therefore, based on these factors the surficial cracking observed during utilization of the pump truck was likely associated with surficial compaction of the imported sand and gravel and not associated with a failure of the basin side slope, in our opinion.

CLOSURE

This report was prepared for the exclusive use of Kiewit-General and the design team for specific application to this project. The letter report is provided for information of factual data only, and not as a warranty of subsurface conditions, such as those interpreted from the exploration logs and discussions of subsurface conditions included in this report.

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist. We assume that the explorations made for this project are representative of the subsurface conditions throughout the site; i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

Within the limitations of the scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practices in this area at the time this report was prepared. We make no other warranty, either express or implied.

If you have any questions regarding the contents of this letter, please contact me at (206) 695-6832.

Sincerely,

SHANNON & WILSON, INC.

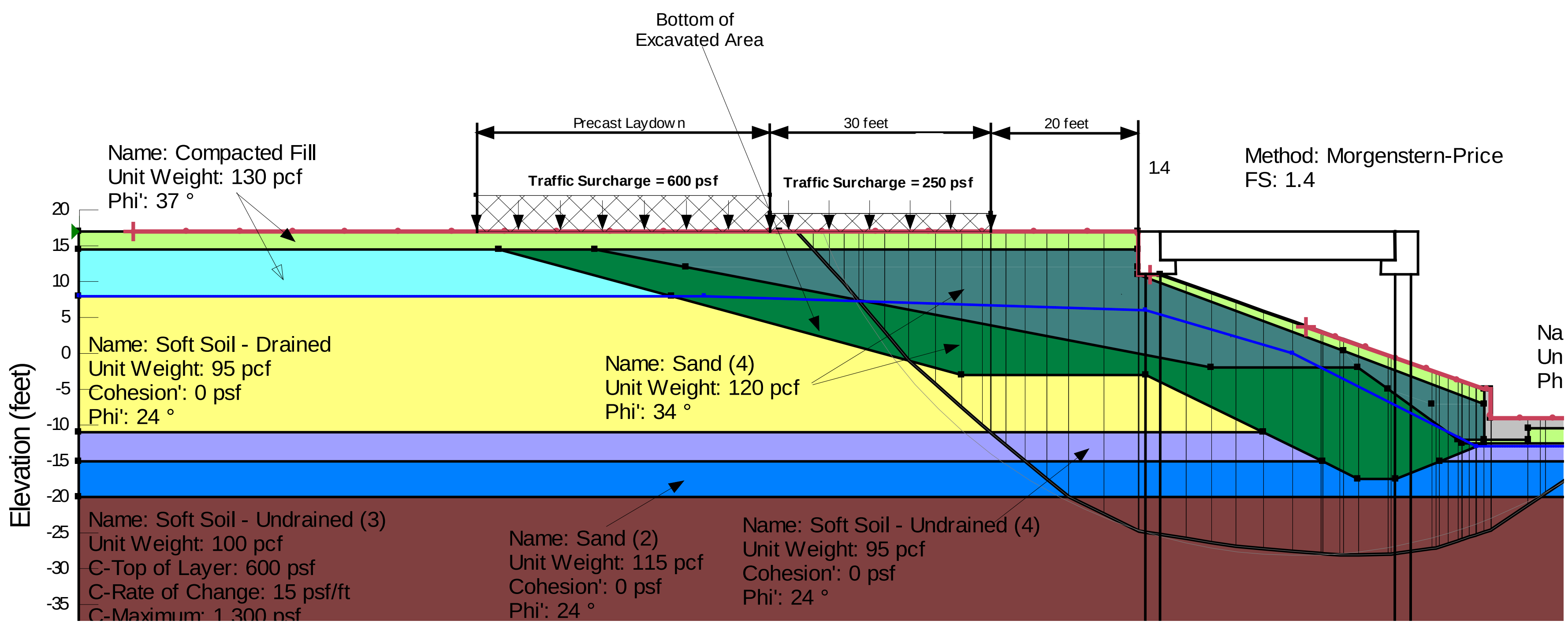
ROBERT A. MITCHELL
STATE OF WASHINGTON
35200
REGISTERED PROFESSIONAL ENGINEER

6/28/13

Robert A. Mitchell, P.E.
Senior Associate

RAM:GJB/ram

Enc. Figure 1 Global Stability Analysis Drained Strength Results Rebuilt Southeast Basin Slope



NOTES:

1. Thick failure surface line corresponds to the critical optimized failure surface. Thin failure surface line corresponds to the critical circular failure surface.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

GLOBAL STABILITY ANALYSES
DRAINED STRENGTH RESULTS
REBUILT SOUTHEAST BASIN SLOPE

June 2013 | 21-1-21190-300

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1

APPENDIX III:
Basin Groundwater Levels

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

ALASKA
CALIFORNIA
COLORADO
FLORIDA
MISSOURI
OREGON
WASHINGTON

June 28, 2013

Mr. Will Morgan
Kiewit-General
1301 West Heron Street
Aberdeen, WA 98520

RE: OBSERVED GROUNDWATER LEVELS AND SLOPE STABILITY, STATE ROUTE (SR) 520 PONTOON CASTING FACILITY, ABERDEEN, WASHINGTON

Dear Mr. Morgan:

This letter compares the groundwater elevations observed during pontoon float out to those assumed during design of pontoon casting facility (PCF). Additionally this letter discusses the groundwater levels observed during design, construction of the basin, and groundwater elevations at two recent seeps in the southwest corner and their relation to basin slope stability.

Figure 1 shows the approximate locations of the vibrating wire piezometers (VWP) that were installed under the centerline of the basin slab (VWP-1 through VWP-5). A VWP was installed in each location at an approximate elevation of -20 feet and -50 feet. These VWPs, termed herein as shallow and deep VWP, were used to observe groundwater levels in the upper aquifer (between elevations -10 and -20 feet) and lower aquifers (below elevation -50 feet).

GROUNDWATER LEVELS DURING FLOAT OUT

Figure 2 shows baseline groundwater elevation data collected from VWP-1, VWP-2, VWP-3, and VWP-5, along with the predicted tide elevations from mid-June through late July 2012, prior to the first float out cycle. The groundwater observed at the shallow and deep VWP-1 closely responded to tidal variation. The deep VWP-2 and the shallow VWP-3, show muted response to tidal variation. The remaining VWPs (shallow VWP-2, deep VWP-3, shallow VWP-5, and deep VWP-5) do not appear to respond to tidal variation. We note that we did not collect data from VWP-4 prior to or during float out cycles, as the VWPs and/or VWP cables in this location were destroyed and/or buried.

In our 2011 Released for Construction Geotechnical Report in order to reduce the potential of base instability during float out, we recommended slowing unwatering to allow time for sufficient groundwater drawdown in the event that groundwater levels in the upper and/or lower aquifer were not sufficiently lowered. We estimated that if a groundwater elevation of about +10 feet or greater was measured in the lower aquifers while the basin was empty during unwatering,

400 NORTH 34th STREET – SUITE 100
PO BOX 300303
SEATTLE, WA 98103
206-632-8020 FAX 206-695-6777
TDD: 1-800-833-6388
www.shannonwilson.com

SHANNON & WILSON, INC.

the factor of safety against hydrostatic uplift on the basin slab could be less than one indicating the potential for base instability.

Figures 3 and 4 show the groundwater elevation data collected from the shallow and deep VWP-1, VWP-2, VWP-3, and VWP-5 during basin flooding and unwatering for the first two float out events in July 2012 and April 2013 respectively. The data shown in Figures 3 and 4 indicate that groundwater levels in the shallow and deep VWPs (VWP-1, VWP-2, VWP-3, and VWP-5) rise and fall with the basin water levels during flooding and unwatering and do not remain elevated following unwatering and were significantly below elevation +10 feet. In our opinion, during the first two float out events the PCF under-slab dewatering system maintained groundwater elevations below levels at which there could be a potential for base instability.

SOUTHWEST CORNER SLOPE SEEPAGE

In general, the groundwater at the site flows through the wood fill layer and down the slope and is intercepted by the trench cutoff drain (invert elevation approximately +4 feet). The contact of the wood fill layer with the underlying relatively impervious clayey silt layer is variable. Therefore, any groundwater not captured by the trench cutoff drain continues down the slope along the cut slope/sand drainage blanket interface and then is collected in the toe wall drain (invert elevation approximately -10 feet).

Kiewit-General (KG) observed the water seepage flowing over the toe wall near the southwest corner of the basin on May 22, 2013. The seepage was observed approximately 20 days after flooding and unwatering of the basin for the second pontoon float out. We understand that KG has not been casting concrete at the southwest precast laydown yards recently, and no process water was generated by construction activities in the southwest corner.

During our May 23rd site visit we observed two to three seeps on the slope in the southwest corner of the basin. The elevation of the highest seep was between approximately +2 and +4 feet. The water from these seeps was flowing over the top of the toe wall (Elevation -4 feet) at a rate of about 4 gallons per minute during high tide. Site observations from KG indicated that the flow of water over the toe wall stopped during low tide.

Based on the seepage location and our observations, it is our opinion that the seepage water could be flowing from either an impacted trench cutoff drain or through a gap in the seepage cutoff wall. The seep would then flow down the slope and flow over or pool behind the toe wall. The rate of water flow over the toe wall would vary with the tide.

SLOPE STATIC FACTOR OF SAFETY

Figure 5 provides the global stability analysis for the static long-term condition results using drained strength properties for the site soils and typical surcharge loading. The global stability analysis considered a groundwater elevation corresponding to the design elevation (+8 feet away from the slope) and groundwater exit elevation on the slope at +2 and +4 feet. The two groundwater exit elevations were proposed to consider potential variations based on the elevation of the two seeps observed in the southwest corner of the PCF.

In our 2011 Released for Construction Geotechnical Report, the design groundwater elevation away from the basin slopes was +8 feet. This groundwater elevation is consistent with the static groundwater elevation measured during the two pumping tests performed at the site and presented in Appendix H of our 2011 Released for Construction Geotechnical Report. Observations the groundwater elevation made during construction of the basin are consistent with the design groundwater elevation of +8 feet away from the basin slopes.

This variation of the groundwater exit elevation (+2 and +4 feet) did not change the resulting factor of safety as shown on Figure 5. Therefore based on this analysis, the static factor of safety for the basin slope is greater than 1.3. Our soil shear strength, groundwater levels, and surcharge loading that was utilized in the slope stability analysis shown on Figure 5, in our June 9 and August 12, 2011 letters, and in our 2011 Released for Construction Geotechnical Report is appropriate and would provide suitable static factors of safety as required by the WSDOT Geotechnical Design Manual.

CLOSURE

This report was prepared for the exclusive use of Kiewit-General and the design team for specific application to this project. The letter report is provided for information of factual data only, and not as a warranty of subsurface conditions, such as those interpreted from the exploration logs and discussions of subsurface conditions included in this report.

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist. We assume that the explorations made for this project are representative of the subsurface conditions throughout the site; i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

Within the limitations of the scope, schedule, and budget, the analyses, conclusions, and recommendations presented in this report were prepared in accordance with generally accepted professional geotechnical engineering principles and practices in this area at the time this report was prepared. We make no other warranty, either express or implied.

If you have any questions regarding the contents of this letter, please contact me at (206) 695-6832.

Sincerely,

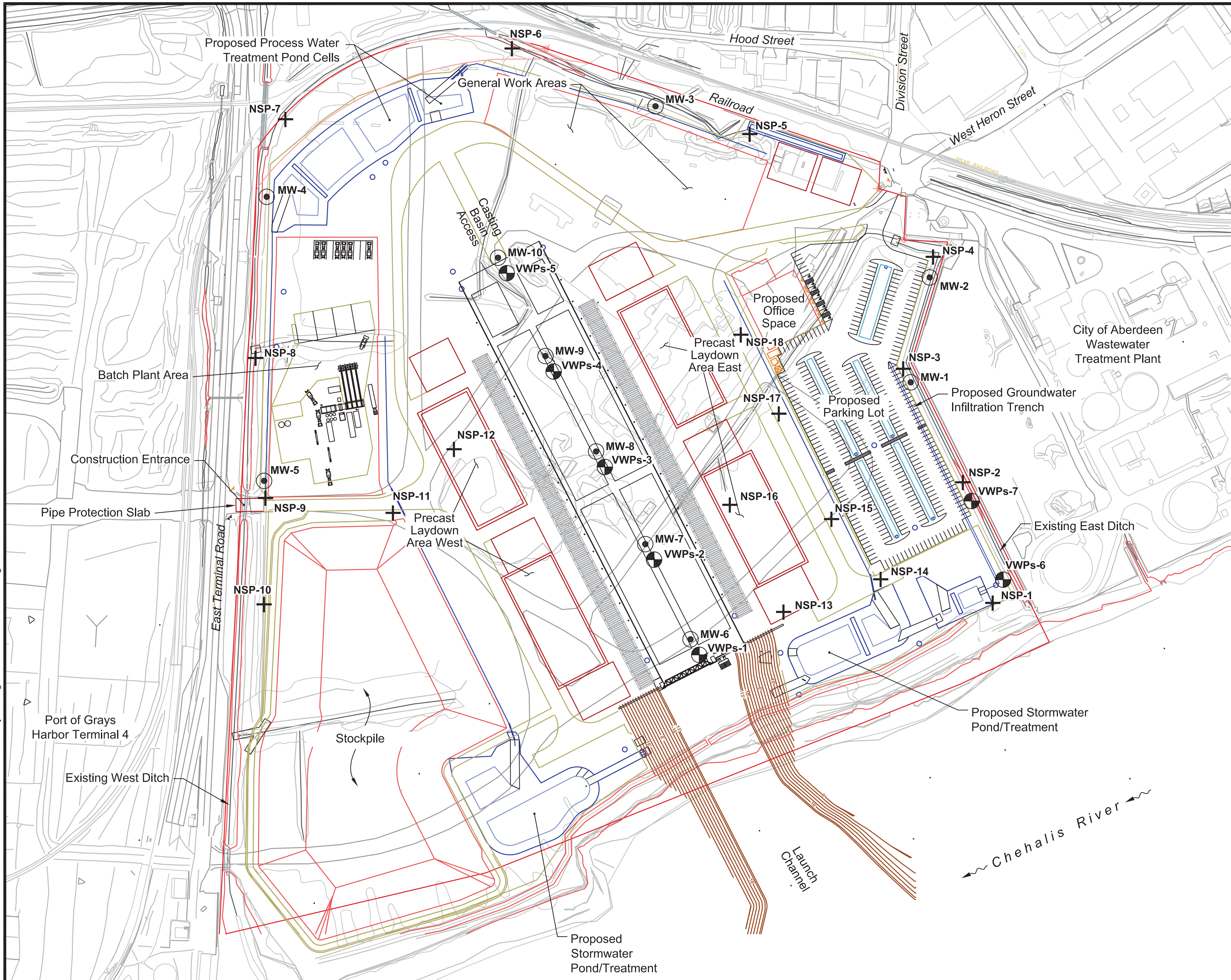
SHANNON & WILSON, INC.

Robert A. Mitchell, P.E.
Senior Associate

RAM/ram

Enc. Figure 1 Geotechnical Instrumentation Layout
Figure 2 Baseline Groundwater Elevations in Centerline Vibrating Wire Piezometers
Figure 3 Float Out Cycle 1 July 2012 Approximate Water Level Elevations
Figure 4 Float Out Cycle 2 April 2013 Approximate Water Level Elevations
Figure 5 Global Stability Analysis Basin Slope 2.5H:1V

Filename: J:\211\21190-015\21-1-21190-015 Instrumentation Layout.dwg Date: 01-10-2011 Login: bac



LEGEND

VWPs-1 — Proposed Vibrating Wire Piezometers (Shallow and Deep)

MW-1 — Proposed Groundwater Monitoring Well

NSP-1 — Proposed Near Surface Settlement Point

NOTES

1. This figure is based on *XL2672_AL_BP_Base Map-Default.dwg* and, *Figure 3-10.dwg*, received 5-10, and *49854_GEO_s_001.dgn*, received 7-26-10.
2. Where practical, install near-surface settlement points on surface electrical vaults and panels.
3. Locations of instrumentation are approximate. Instruments will be located in the field.

0 200 400

Scale in Feet

SR 520 Pontoon Casting Facility
Aberdeen, Washington

GEOTECHNICAL INSTRUMENTATION LAYOUT

December 2010 21-1-21190-015

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. 1

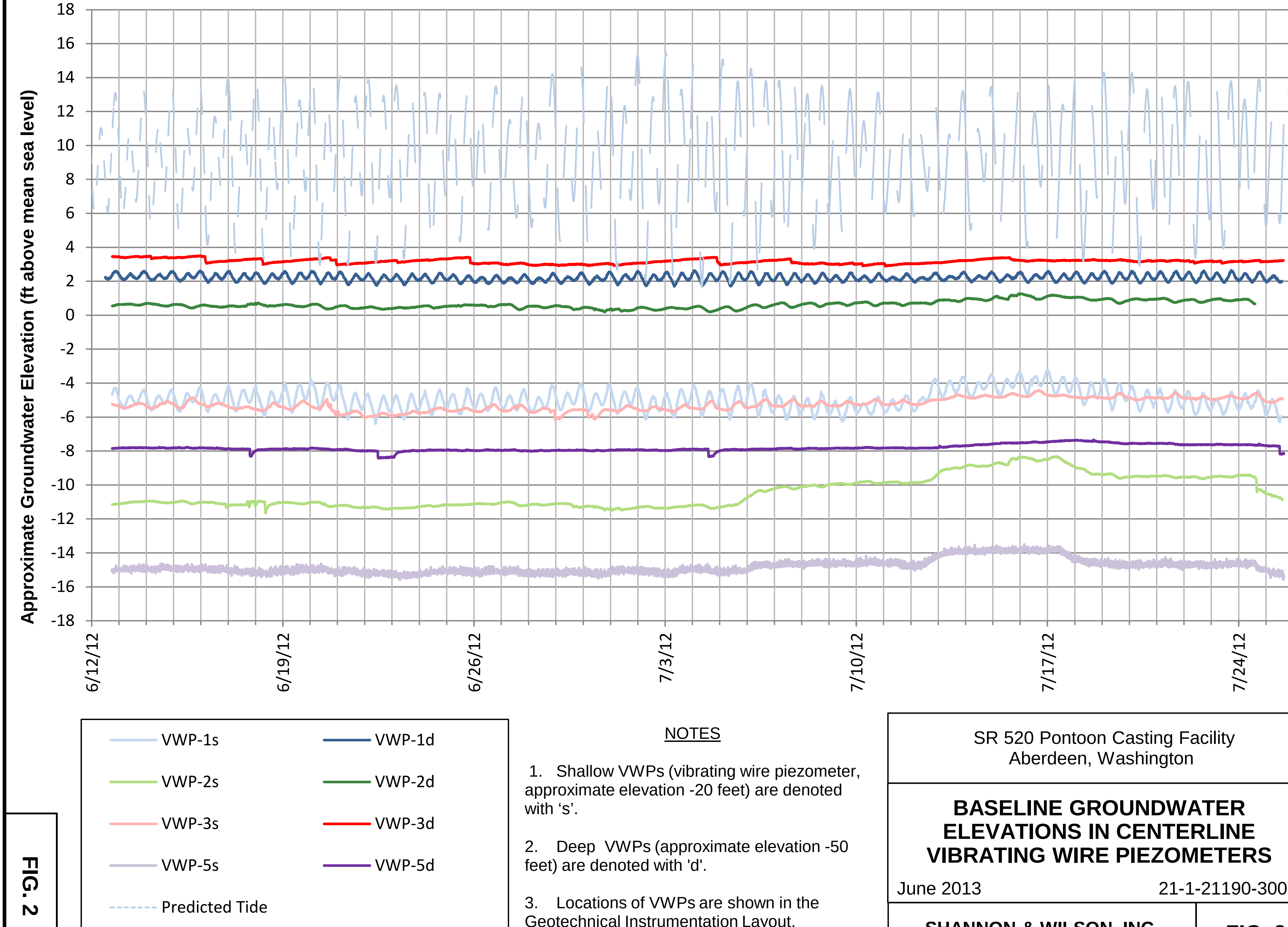


FIG. 2

NOTES

1. Shallow VWPs (vibrating wire piezometer, approximate elevation -20 feet) are denoted with ‘s’.

2. Deep VWPs (approximate elevation -50 feet) are denoted with 'd'.

3. Locations of VWPs are shown in the Geotechnical Instrumentation Layout.

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BASELINE GROUNDWATER ELEVATIONS IN CENTERLINE VIBRATING WIRE PIEZOMETERS

June 2013 21-1-21190-300

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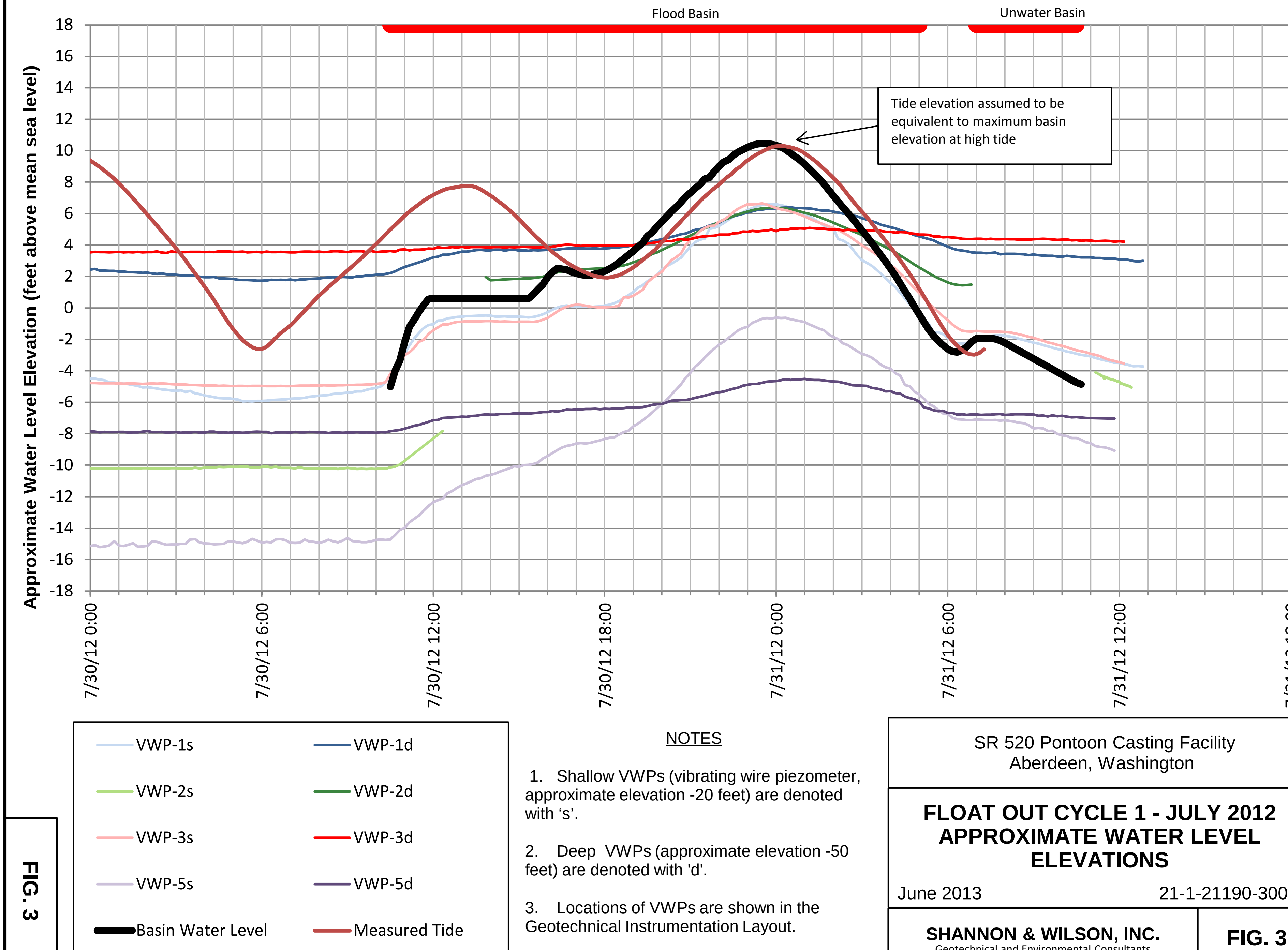
FIG. 2



NOTES

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2. Deep VWPs (approximate elevation -50 feet) are denoted with 'd'.

3. Locations of VWPs are shown in the Geotechnical Instrumentation Layout.

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FLOAT OUT CYCLE 1 - JULY 2012 APPROXIMATE WATER LEVEL ELEVATIONS

June 2013 | 21-1-21190-300

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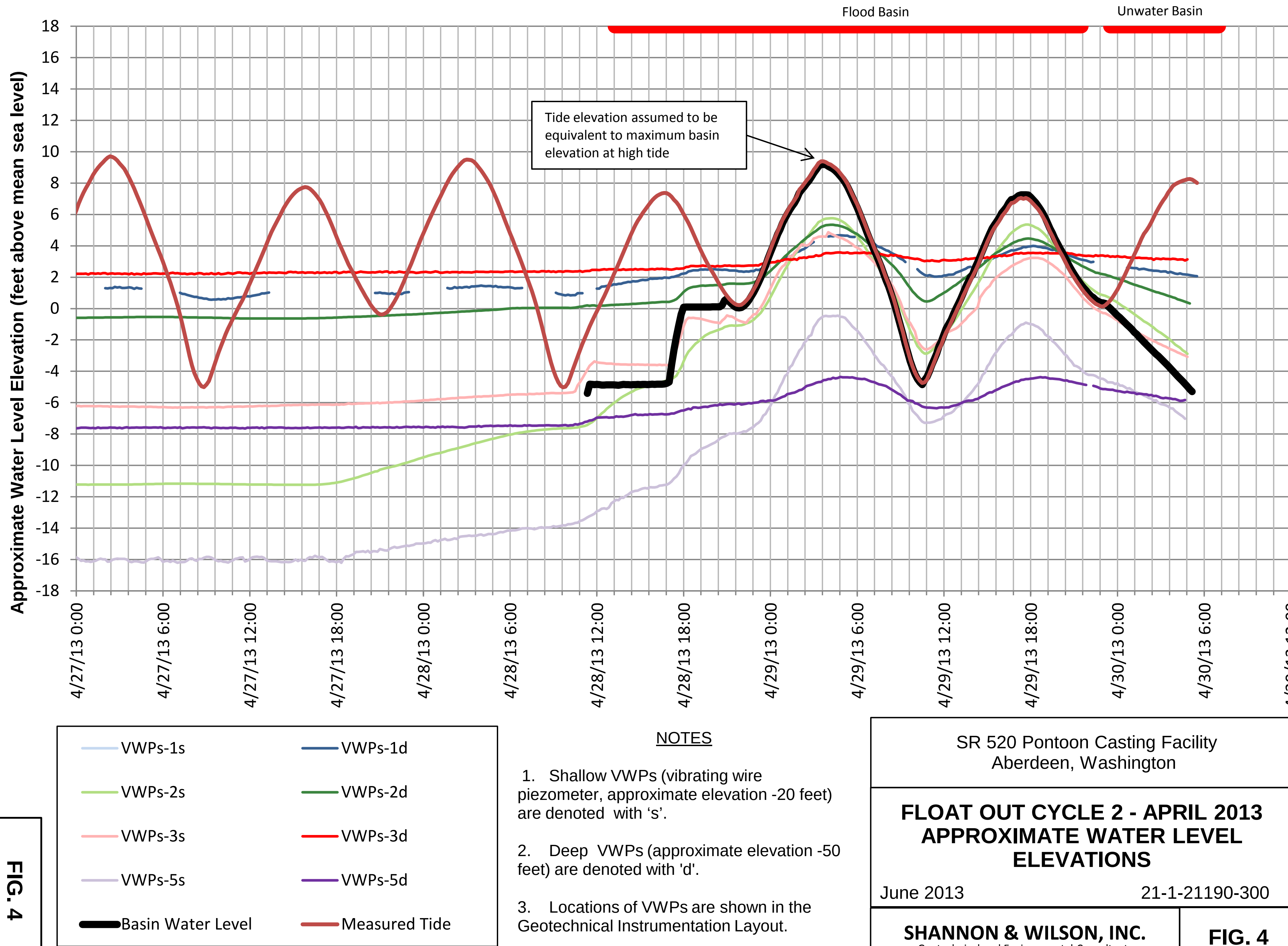
FIG. 3

FIG. 3



NOTES

1. Shallow VWPs (vibrating wire piezometer, approximate elevation -20 feet) are denoted with 's'.

2. Deep VWPs (approximate elevation -50 feet) are denoted with 'd'.

3. Locations of VWPs are shown in the Geotechnical Instrumentation Layout.

SR 520 Pontoon Casting Facility
Aberdeen, Washington

FLOAT OUT CYCLE 2 - APRIL 2013 APPROXIMATE WATER LEVEL ELEVATIONS

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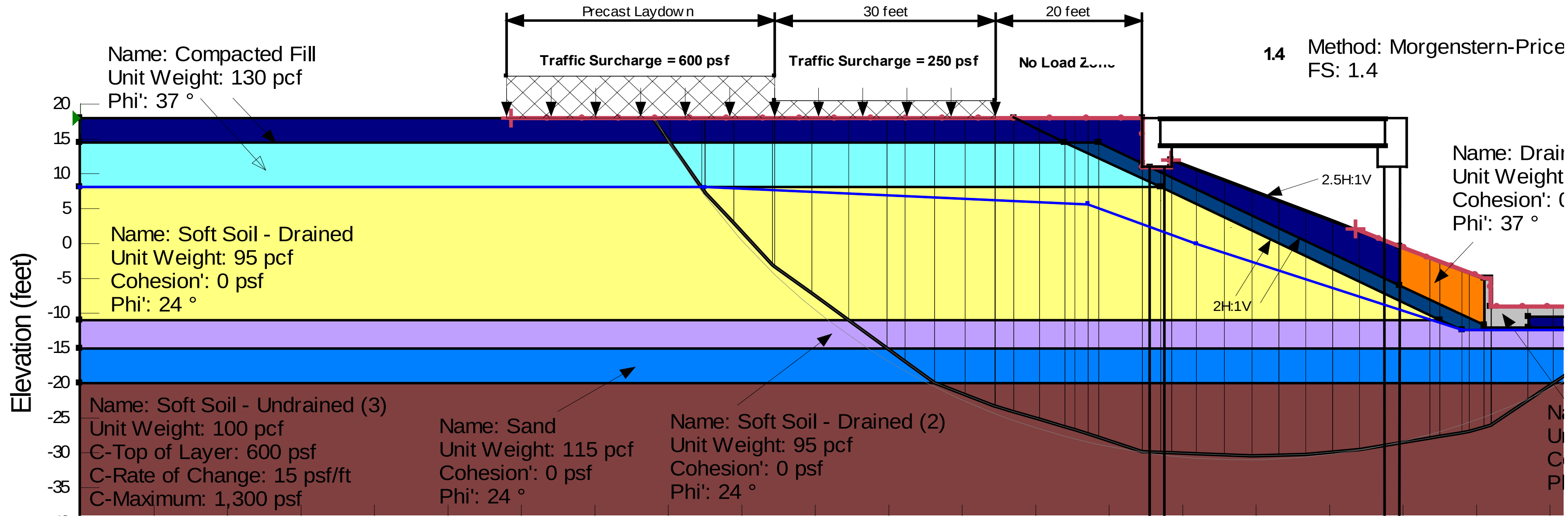
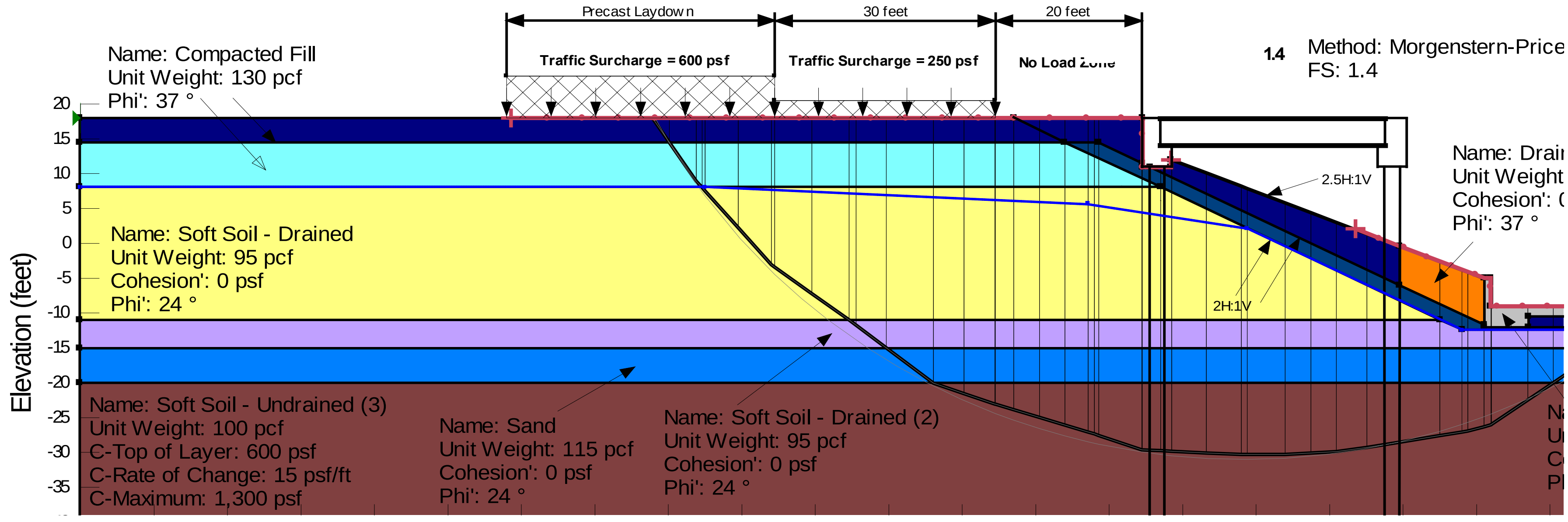
FIG. 4

FIG. 4

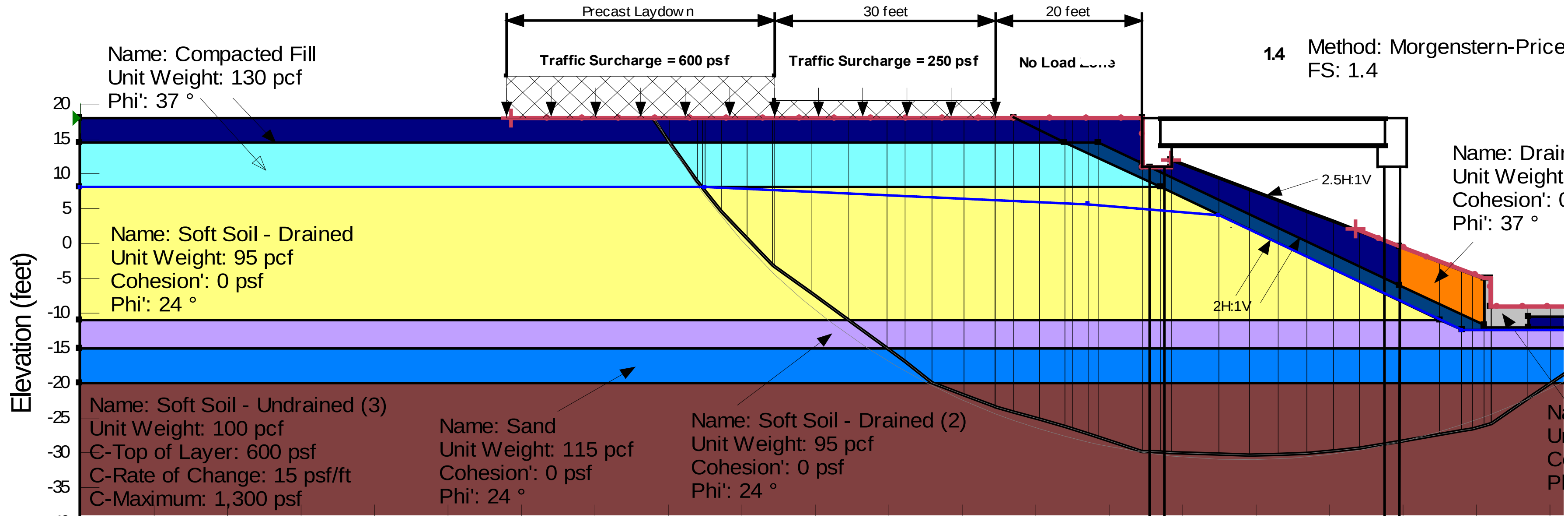
DESIGN GROUNDWATER ELEVATION



GROUNDWATER CONTACT SLOPE FACE AT ELEVATION 2 FEET



GROUNDWATER CONTACT SLOPE FACE AT ELEVATION 4 FEET



NOTES:

1. Thick failure surface line corresponds to the critical optimized failure surface. Thin failure surface line corresponds to the critical circular failure surface.

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GLOBAL STABILITY ANALYSES RESULTS BASIN SLOPE 2.5H:1V

June 2013 21-1-21190-015

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FIG. 5

FIG. 5

APPENDIX IV:
Geotechnical Instrumentation Plan

Below is a table of threshold and limiting values in relation to the Near Surface Settlement Points (NSPs);

Instrument Type	**Threshold-Limiting Value**	**Threshold Monitoring Frequency**
Near Surface Settlement Points	*Vertical Displacement*	*Monitoring Frequency*
NSP-1 through NSP-8	0.1-0.2 FT	Annually each January
NSP-9 through NSP-10	0.25-0.5 FT	Annually each January
NSP-10 through NSP-18		Destroyed during basin construction, no longer require monitoring.

1.1.2 Seismographs

Seismographs are instruments that measure vibration intensity and frequency. Vibration levels from construction activities will be monitored at structures located within 100 feet of the area where construction activity is occurring. In general, vibrations will be monitored during the installation of the pipe piles and any sheet piles that are installed with either impact or vibratory hammers, or other construction activities that generate significant vibrations.

Vibrations will be measured in terms of frequency and peak particle velocity (PPV). During construction, seismographs will be placed at the ground surface adjacent to each structure to determine that vibration levels are below the response values. Background vibrations will be recorded for each adjacent structure and at representative ground locations before the start of construction. The response values for allowable PPV will be coordinated with utility and/or structure owners. The magnitude of the response values will consider the nature of the facility, the type of construction, and its existing condition.

Below is a table of threshold and limiting values in relation to the Vibration Monitors and PPV;

Instrument Type	**Threshold-Limiting Value**	**Threshold Monitoring Frequency**
Vibration Monitors	*Peak Particle Displacement*	*Monitoring Frequency*
	0.5-1 in./sec	Once daily within 100 feet of pile or sheet driving activities, during construction.

1.1.3 Monitoring Wells (MWs)

Monitoring wells (MWs) and VWPs obtain groundwater level measurements associated with the dewatering operations. The locations of the MWs and VWPs are shown in Figure 39. The primary purpose of the MWs and VWPs is to observe groundwater drawdown around the site for correlation to settlement observed by the NSPs. The groundwater measurements will also provide an early indication of future potential ground settlements. That is, the pore pressure changes will generally occur before ground settlement would be observed, considering the fine-grained nature of the foundation soils. Additionally, the VWPs beneath the basin slab would be permanent and used to observe pore pressure during the unwatering cycles. Dataloggers can be connected to the VWPs, and water level loggers can be installed in the MWs to obtain groundwater level readings at closely spaced time intervals without the need for manual surveying. KG may install dataloggers and water level loggers in select MWs or VWPs near settlement sensitive facilities. Although a monitoring frequency is listed for MW-1 through MW-5, they have been decommissioned and no longer require monitoring.

Below is a table of threshold and limiting values in relation to the Vibrating Wire Piezometers (VWPs) and Monitoring Wells (MWs);

Instrument Type	**Threshold-Limiting Value**	**Threshold Monitoring Frequency**
Monitoring Wells	*Change in Elevation*	*Monitoring Frequency*
MW-1	6-7 FT below historic low	Weekly until the construction of the basin is completed
MW-2	5-6 FT below historic low	Weekly until the construction of the basin is completed
MW-3 through MW-5	8-9 FT below historic low	Weekly until the construction of the basin is completed
Vibrating Wire Piezometers		
VWP 1-5 (shallow)	The groundwater pressure head (ft) shall be equal to or below each excavation level prior to the excavation of the level. For overwatering during flooding and unwatering the water level shall be equal to or below the water level surface.	Hourly during unwatering cycles

VWP 1-5 (deep)	The groundwater pressure head (ft) shall be equal to or less than 1.35 times the thickness of soil (ft) above the VWP position. For overwatering during flooding and unwatering the groundwater pressure head shall be equal to or less than 1.35 times the thickness of the soil (ft) above the VWP position plus the height of water in the basin	Hourly during unwatering cycles
VWP-6 (shallow)	1-2 FT below historic low	When construction has been completed and the permanent dewatering system is functioning, the monitoring frequency can be increased to once during unwatering cycles.
VWP-6 (deep)	3-4 FT below historic low	When construction has been completed and the permanent dewatering system is functioning, the monitoring frequency can be increased to once during unwatering cycles.
VWP-7 (shallow)	3-4 FT below historic low	When construction has been completed and the permanent dewatering system is functioning, the monitoring frequency can be increased to once during unwatering cycles.

VWP-7 (deep)	5-6 FT below historic low	When construction has been completed and the permanent dewatering system is functioning, the monitoring frequency can be increased to once during unwatering cycles.

1.2 Monitoring Frequency

Monitoring frequency will vary widely for each of the instrument systems and for each category of construction. DMPs, MWs/VWPs, and seismographs will be installed and a minimum of four readings, ideally at least one week apart, will be obtained before the start of construction to provide a baseline.

A typical monitoring frequency for DMPs is once-daily visual monitoring of points within 100 feet of pile driving operations. The visual monitoring, performed by KG, will include observations such as ground and/or structure cracking, gradual ground depressions and slopes, pavement cracking or settlement, and similar indications of ground, structure, and pavement distress.

When construction has been completed and the permanent dewatering system is functioning, the monitoring frequency can be increased to once a month depending upon the results of the MWs and VWPs readings. If groundwater levels and pressures continue to change over the one-month period, the frequency of the survey measurements will be increased to weekly as determined by KG. All DMPs monitored during pile driving will be monitored at least weekly until those operations are complete.

There will be continuous seismograph monitoring for vibration-causing activities within 10 feet of cast-iron water mains, within 20 feet of other pipelines, and within 100 feet of other structures.

All MWs and VWPs will be monitored weekly until the construction of the basin is completed. Some of the VWPs are temporary for use during construction. The VWPs beneath the basin slab are permanent and used to observe pore pressure during the unwatering cycles. The VWPs beneath the basin slab will be monitored on an hourly basis during unwatering cycles.

1.3 Response Values

Response values will be established for structures, utilities, and other critical features prior to the start of construction. These response values are based on the condition of the structures and utilities and the baseline monitoring data. The response values typically include “threshold” and “limiting” values. The threshold values represent a level of movement that warrants attention. If the instruments indicate that the threshold values have been experienced, remedial measures will be prepared in order to mitigate the vibration, movement, or adverse pore pressure changes that are occurring. Threshold values are typically some percentage of limiting values. If the instruments indicate that the limiting value has been experienced, remedial measures will be implemented immediately or construction suspended to prevent adverse impacts to the structures being monitored.

1.4 Data Reduction and Reporting

Baseline measurements will be obtained prior to the beginning of construction. Baseline data is used for establishing response values and assessing the need for implementing mitigation measures, as well as for resolving potential disputes, especially with respect to the impacts of construction on adjacent structures.

Since the collected and reduced data may be critical to assessing performance, all data must be reported within a few hours. Therefore, QC will verbally share that data with the Engineer of Record and QA within eight hours of the readings being collected.

Due to the quantities of data that could be collected on a daily basis, only the values that exceed 75% of the limiting value, or meet the limiting value need to be reported by KG to the engineer of record. KG will report this information directly to the Engineer of Record. The communication will include a summary of the construction activities performed during the monitoring period in the vicinity of the instrumentation. Also included will be a corrective action plan to mitigate the cause of the displacement, vibration, or groundwater level. This plan will be consistent with what is currently used on the project, containing the following steps: Root cause analysis, repair, correction for issue, and action plan to prevent reoccurrence. In the event that 100% of the limiting value is reached, the operation causing the disturbance will be temporarily suspended, and the previously listed steps will be followed to mitigate the issue, and allow work to resume. This communication will allow the PCF to perform as designed.

In order to streamline the data sharing process and reduce closeout documentation, any monitoring data collected will be summarized by QC and submitted to the owner within 60 days of project physical completion.

KIEWIT-GENERAL GEOTECHNICAL INSTRUMENTATION PLAN

1.1 Geotechnical Instruments

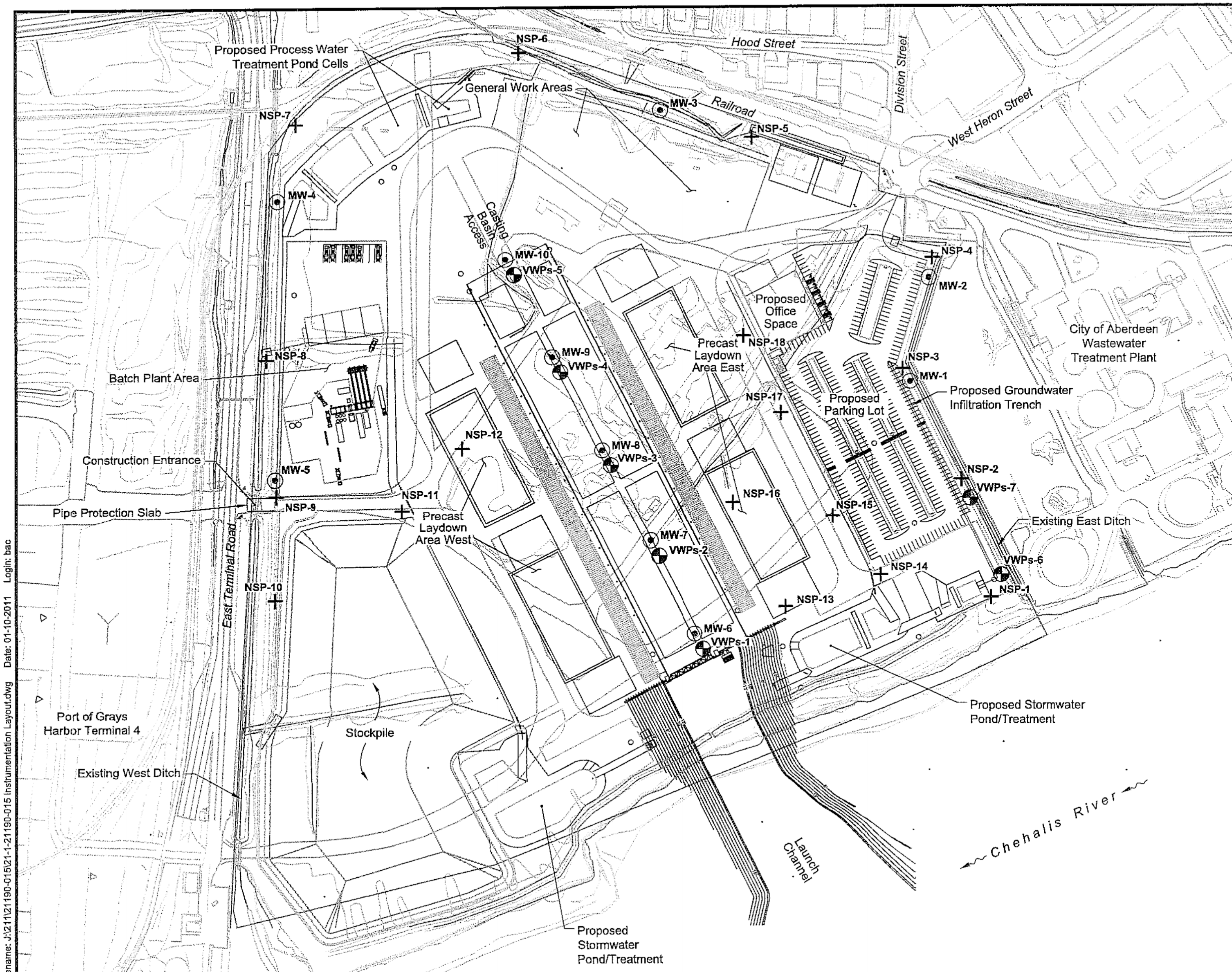
The types, numbers, and locations of the geotechnical instruments depend on the construction methods, sequence, and durations, as well as on the proximity, foundations characteristics, and conditions of adjacent facilities. The instrument types discussed in the following sections will be used in the geotechnical instrumentation and monitoring plan. The geotechnical instrumentation is discussed below and the layout is shown in Figure 39.

1.1.1 Deformation Monitoring Points (DMPs)

Deformation monitoring points (DMPs) are fixed markers (survey hubs, pins, or targets) monitored (in conjunction with standard surveying techniques) to evaluate vertical and horizontal deformations. DMPs are an effective method of monitoring ground and adjacent facility movements to assist with assessing construction-induced impacts. DMPs include near-surface settlement points placed near the ground surface for the purpose of monitoring changes in elevation of existing ground. All settlement points will be monitored by optical or laser survey methods to determine displacements.

Near-surface settlement points (NSPs) consist of settlement rods driven into place to ensure that the rods will move with the soil in which they are embedded. Each settlement rod is protected by a warning stake or bollard to prevent damage from construction traffic. In conjunction with survey equipment, NSPs are used to monitor settlements in unimproved areas, settlement associated with dewatering, and locations adjacent to settlement sensitive structures. Locations for the NSPs are shown in Figure 39. NSPs will be located adjacent to proposed project features that will provide a barrier from construction traffic (i.e., vaults, light poles), such that they will not be disturbed as construction proceeds.

All DMPs and NSPs will be monitored by optical or laser survey methods annually with any displacements recorded. At the close of the project a summary of all DMP and NSP displacements will be completed and given to WSDOT for future reference.



N

LEGEND

VWPs-1 — Proposed Vibrating Wire Piezometers (Shallow and Deep)

MW-1 — Proposed Groundwater Monitoring Well

NSP-1 — Proposed Near Surface Settlement Point

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0 200 400

Scale in Feet

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Aberdeen, Washington

GEOTECHNICAL INSTRUMENTATION LAYOUT

December 2010 21-1-21190-015

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FIG. 39